Making the Case for Tailored Stormwater Management

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ABSTRACT

Protection of downstream channels and reduction in flooding can potentially be improved by evaluating alternative site stormwater management (SWM) strategies at a watershed scale and selecting the optimal strategy for a subject watershed. Tailoring a management strategy for a specific watershed may be worthwhile to minimize development costs and maximize downstream benefit. A hydrologic/hydraulic model for a watershed in Blacksburg, Virginia, is used to evaluate downstream results based on implementation of several alternative SWM strategies currently practiced within the United States.

Results show none of the strategies meet the goal of maintaining the baseline goal at the watershed POI for the full range of design storms. Modification to the strategy that performs best at the watershed scale did meet the watershed goal for all design storms except the 1-year. For smaller storm events, it appears that increasing the volume of an initial capture and the drawdown time to release that volume does not increase performance downstream. This is potentially significant as extra dollars spent on site would not provide extra benefit downstream. When post-development peak runoff rates are detained to the predevelopment rate for larger storm events, whether based on a site or watershed focused strategy, the watershed goal can be met. A volume reduction strategy performs well, but implementation is hindered by soils with poor infiltration and the presence of karst.

Other insight to watershed based management strategies, the role of regional facilities and predevelopment condition assumptions at the site scale to maintain a baseline condition downstream are discussed.

Dedication

I dedicate this thesis to Dr. G.V. Loganathan, Brian Bluhm and Matthew Gwantley, each of whom passed on April 16, 2007. These men shared my interest in the subject matter and each represents the excellence in character I admire in our field of study.

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Table of Contents

1 Introduction ... 1-1
1.1 Problem Statement ... 1-1
1.2 Research Objectives ... 1-2
2 Literature Review and Background Information ... 2-1
2.1 Stormwater Management Strategies at the Watershed Scale ... 2-1
2.2 Traditional Stormwater Management ... 2-3
2.2.1 Maintaining Predevelopment Peak Runoff Rate ... 2-3
2.2.2 Treatment component added to peak rate reduction ... 2-6
2.3 Alternative Stormwater Management ... 2-7
2.3.1 Downstream peak focus *blanket* strategy ... 2-8
2.3.1.1 Unit Release Rates - Milwaukee ... 2-8
2.3.1.2 Standardized Release Rate – Overland Park ... 2-9
2.3.2 Downstream peak focus *stratified* strategy ... 2-11
2.3.2.1 Stratified release rate - development ... 2-11
2.3.2.2 Stratified release rate - Implementation ... 2-14
2.3.3 Volume focus strategy ... 2-16
2.3.3.1 Low Impact Development ... 2-16
2.3.3.2 Full Spectrum Detention ... 2-20
2.3.3.3 Volumetric Design Procedure ... 2-21
3 Methodology: Model Development & Strategy Representation ... 3-1
3.1 Watershed Model Development ... 3-1
3.1.1 Subject Watershed ... 3-1
3.1.2 Watershed Data ... 3-3
3.1.2.1 Hydrology - Curve Number ... 3-5
3.1.2.2 Hydrology - Time of Concentration ... 3-6
3.1.2.3 Hydrology - Rainfall ... 3-7
3.1.2.4 Stormwater Conveyance System ... 3-8

3.1.2.5 Stormwater Management Facilities ... 3-9
3.1.3 Model Calibration ... 3-9
3.2 Modeling Scenarios & Application of Alternative Strategies ... 3-12
3.2.1 Applicable Areas ... 3-12
3.2.2 Existing Regional Facilities ... 3-12
3.2.3 Strategy Parameter Development ... 3-13
3.2.3.1 Baseline Development ... 3-13
3.2.3.2 Unit Release Rate ... 3-16
3.2.3.3 Stratified Release Rate Approach ... 3-18
3.2.3.4 Volumetric Design Procedure ... 3-21
3.2.4 Stormwater Management Modeling Scenarios ... 3-22
3.3 Alternative Strategy Model Representation ... 3-23
3.3.1 Traditional Stormwater Management ... 3-24
3.3.1.1 Maintaining Predevelopment Peak Runoff Rate ... 3-24
3.3.1.2 Treatment Component Added to Peak Rate Reduction ... 3-25
3.3.2 Downstream peak Focus Blanket Strategy ... 3-27
3.3.2.1 Unit Release Rate ... 3-27
3.3.3 Downstream peak Focus Stratified Strategy ... 3-28
3.3.3.1 Stratified Release Rates ... 3-28
3.3.4 Volume Focus Strategy ... 3-30
3.3.4.1 Low Impact Development ... 3-30
3.3.4.2 Full Spectrum Detention ... 3-32
3.3.4.3 Volumetric Design Procedure ... 3-35
4 Modeling Results ... 4-1
4.1 Runoff at the Site Scale ... 4-1
4.1.1 Strategy variations ... 4-1
4.1.2 Site runoff characteristics ... 4-3
4.1.2.1 Traditional ... 4-5
4.1.2.2 Traditional with Water Quality Component ... 4-5
4.1.2.3 Unit Release Rates ... 4-6

4.1.2.4 Stratified Release Rates 4-7
4.1.2.5 Low Impact Development 4-7
4.1.2.6 Full Spectrum Detention 4-8
4.1.2.7 Volumetric Design Procedure 4-9
4.2 Watershed Runoff Characteristics 4-9
4.2.1 Watershed characteristics 4-11
4.2.1.1 Applicable Areas 4-11
4.2.1.2 Critical locations 4-12
4.2.1.3 Runoff Response Timing 4-15
4.2.2 Strategy correlation at a watershed scale 4-15
4.2.2.1 Traditional 4-15
4.2.2.2 Traditional with Water Quality Component 4-16
4.2.2.3 Unit Release Rate 4-17
4.2.2.4 Stratified Release Rates 4-19
4.2.2.5 Low Impact Development 4-20
4.2.2.6 Full Spectrum Detention 4-21
4.2.2.7 Volumetric Design Procedure 4-22
4.2.2.8 Supplementary Model Scenarios 4-24
4.2.3 Other considerations 4-25
4.2.3.1 Volume requirements at the site scale 4-25
4.2.3.2 Implementation constraints 4-28
5 Tailored Stormwater Management 5-1
5.1 Management Strategy Evaluation, Selection and Modification 5-1
5.1.1 Results evaluation 5-2
5.1.2 Evaluation of Emerson's shortcomings 5-4
5.1.3 Strategy Selection 5-6
5.1.4 Tailored Management for the Subject Watershed 5-7
5.2 Tailored Management Guidance 5-11
6 Discussion and Conclusions 6-13
6.1 Review of Objectives 6-13

6.2 Final Thoughts ... 6-15
6.3 Future Research ... 6-17
7 References ... 7-1
8 Appendix ... 8-1
Appendix A: LID Chart Series ... 8-1
Appendix B: Maps of Subject Watershed ... 8-5
Appendix C: SewerGEMS Implicit engine option settings ... 8-9
Appendix D: Curve number data for subject watershed (CGIT's CN Development Report ... 8-11
Appendix E: NOAA 24-hour Precipitation Depths ... 8-25
Appendix F: Comparisons between measured and modeled flow data ... 8-27
Appendix G: Mapping of Applicable Areas (A through I) ... 8-30
Appendix H: Baseline (predevelopment), existing conditions and no detention hydrographs ... 8-32
Appendix I: Common information for modeling scenarios ... 8-40
Appendix J: Information for Traditional strategy scenario ... 8-48
Appendix K: Information for Traditional with WQ component strategy scenario ... 8-59
Appendix L: Information for the Unit Release Rate strategy ... 8-77
Appendix M: Information for the Stratified Release Rates strategy ... 8-89
Appendix N: Information for the Low Impact Development strategy ... 8-102
Appendix O: Information for the Full Spectrum Detention strategy ... 8-117
Appendix P: Information for the Volumetric Design Procedure strategy ... 8-134
Appendix Q: Strategy Comparison Hydrographs at the POI ... 8-145
Appendix R: Information for the Tailored Strategy ... 8-154
Appendix S: Watershed Runoff hydrographs for the Tailored Strategy ... 8-165

List of Figures

Figure 2.1: Hypothetical 200-acre watershed. (McEnroe and Heatherman 2006) 2-10
Figure 2.2: Hydrograph analysis example for determining release rate requirements (Lehigh Valley Planning Commission 2000) 2-15
Figure 2.3: Approximate detention basin routing for rainfall Types I, IA, II and III (SCS, 1986) 2-19
Figure 2.4: Visual depiction of the critical time period. (Milwaukee Metropolitan Sewerage District)..2-22
Figure 3.1: Histogram illustrating the distribution of sub-watershed size..................... 3-2
Figure 3.2: Graphical depiction that illustrates the VDP requirement to maintain the predevelopment volume during the critical time period...3-22
Figure 3.3: Establishment of the EURV was generated using data from all applicable areas for a range of design storms...3-34
Figure 4.1: Stage-discharge curve for the Lower Wong park regional facility.............4-14
Figure 5.1: A portion of the watershed POI 2-year design storm hydrographs at, and near, the bank full condition...5-5

List of Tables

Table 2.1 Summary of characteristics examined by Goff and Gentry (2006)…………..2-5
Table 3.1: Precipitation depths from NOAA for specified 24-hour design storms..........3-7
Table 3.2: Summary of structure information within the conveyance system................3-9
Table 3.3: Notable measured rainfall in the subject watershed....................................3-10
Table 3.4: Peak runoff rate and volume comparisons between measured and modeled flow data...3-11
Table 3.5: Baseline (predevelopment) information at the POI for SCS 24-hour design storms...3-14
Table 3.6: Peak runoff rates and runoff volumes at the outfall of the watershed for Scenarios 1 through 3..3-15
Table 3.7: Baseline (predevelopment) information at the regional facility points of interest...3-16

Table 3.8a: Information for application of the determined unit release rates for the 2-year design storm. *Area* reflects area of applicable area; *allowable release rate* is the area multiplied by the URR and *design* value is the actual release rate after modification of the control structure....3-17
Table 3.8b: Information for application of the determined unit release rates for the 25-year design storm. *Area* reflects area of applicable area; *allowable release rate* is the area multiplied by the URR and *design* value is the actual release rate after modification of the control structure....3-18
Table 3.9: Comparison of peak outflow at the watershed POI between the baseline predevelopment peak runoff rates and with the unit release rates applied to the watershed....3-18
Table 3.10: Values used for generation of release rates for applicable areas: 2-year storm....3-19
Table 3.11: Information used for generation of release rates for applicable areas: 10-year storm....3-20
Table 3.12: Information used for generation of release rates for applicable areas: 25-year storm....3-20
Table 3.13: Summary of strategies and their general characteristics....3-24
Table 3.14: Allowable release rates per applicable area for the 2-, 10- and 25-year storms....3-29
Table 3.15: EURV for each applicable area is calculated based on the EURV as inches per acre established in Figure 3.3....3-34
Table 3.16: Allowable release volumes during the critical time period....3-36
Table 4.1: Summary of design parameters for the alternative strategies....4-2
Table 4.2: Comparison of percent differences in peak runoff rates when compared to predevelopment peak runoff rates....4-4
Table 4.3: Average minutes that the post-development peak occurs after the predevelopment peak for each management strategy....4-5
Table 4.4: Summary of peak runoff rates at the watershed outfall based for the alternative strategies....4-10

Table 4.5: Maximum HGL in the Owens Park regional facility with each management strategy implemented throughout the watershed...4-13
Table 4.6: Maximum HGL in the Lower Wong Park regional facility with each management strategy implemented throughout the watershed.....................................4-14
Table 4.7: Summary of peak runoff rates at the watershed outfall for the Traditional strategy...4-16
Table 4.8: Summary of peak runoff rates at the watershed outfall for the Traditional with a Water Quality component strategy...4-16
Table 4.9: Summary of peak runoff rates at the watershed outfall for the Unit Release Rate strategy...4-18
Table 4.10: Summary of peak runoff rates at the watershed outfall for the Stratified Rate strategy...4-19
Table 4.11: Summary of peak runoff rates at the watershed outfall for the Low Impact Development strategy...4.21
Table 4.12: Summary of peak runoff rates at the watershed outfall for the Full Spectrum Detention strategy..4-22
Table 4.13: Summary of peak runoff rates at the watershed outfall for the Volumetric Design Procedure strategy..4-23
Table 4.14: Maximum volume used in management facilities serving each applicable area for each management strategy...4-26
Table 4.15: Percent of volume used in management facilities serving each applicable area for each management strategy compared to volume used with the implementation of the Traditional strategy..4-26
Table 4.16: Relative capital cost estimates for implementing each strategy at each applicable area..4-27
Table 4.17: Percent of cost for management facilities serving each applicable area for each management strategy compared to cost for the implementation of the Traditional strategy...4-28
Table 5.1: Summary of peak runoff rates at the watershed outfall for the Tailored strategy...5-9
Table 5.2: Summary of stages at the watershed outfall for the Tailored strategy...........5-9

Table 5.3: Maximum volume used in management facilities serving each applicable area for Tailored, Traditional with WQ, or Traditional strategy...5-10
Table 5.4: Percent of volume used in management facilities serving each applicable area for Tailored, Traditional with WQ, or Traditional strategy, compared to volume used with the implementation of the Traditional strategy...5-10
Table 5.5: Relative capital cost estimates for implementing the Tailored, Traditional, or Traditional strategy...5-10
Table 5.6: Percent of cost for management facilities serving each applicable area for the Tailored, Traditional, or Traditional strategy compared to cost for the implementation of the Traditional strategy...5-11

1 Introduction

The predominant stormwater management strategy used by most jurisdictions for improving conditions at receiving streams has been on-site detention at the *site* development scale to maintain predevelopment peak flows for specified design storms. This strategy is referred to throughout this report as the Traditional Management Strategy. A more recent strategy includes treatment of the "first flush" to reduce pollutant loads from passing downstream. The addition of the treatment of the first flush can affect runoff response in some cases when runoff volumes are infiltrated or released over an extended period of time. Often municipalities find the need to construct regional stormwater management facilities to mitigate downstream flooding problems in spite of the presence of traditional stormwater management upstream. Planning and installation of these regional facilities are typically reactionary in nature in that available land, funding and flood prone areas dictate design. Although these facilities may alleviate immediate downstream flooding problems, the benefit from the capital investment may be localized in that benefit may not improve conditions in the receiving streams, usually degraded by longer duration of high flows associated with smaller, more frequent storm events.

Although detention can reduce post-development peak flows to predevelopment conditions, maintaining these peaks on a site scale basis may not significantly reduce peak flows in receiving streams due to increased volume from development and the summation of many flat, longer duration hydrographs within the watershed. The Henrico County Environmental Program Manual states that there had been "little to any improvement to degraded stream systems present in the County because the requirements (for stormwater management) were based on the needs of the site, not the needs of the watershed" (Henrico County, 2001).

1.1 Problem Statement

Traditional stormwater management implemented at the site scale does not significantly address flooding problems downstream at a watershed scale. Protection of downstream channels and reduction in flooding can potentially be improved by evaluating alternative stormwater management strategies at a watershed scale. Available GIS data and hydrologic/hydraulic modeling software provide capabilities for improved watershed runoff modeling. To maximize downstream benefit with "tailored" stormwater management (SWM), the watershed should be subjected to a variety of available management strategies. Tailored stormwater management refers to the selection of an optimal management strategy, with possible modification, to be implemented within a specific watershed. It is suggested that this selection should be based from the results of a watershed analysis that examines results at a point, or points, of interest within the watershed. Available stormwater management strategies may be classified to have:

1. blanket requirements, in which all site developments within the watershed are subject to identical release requirements or
2. stratified requirements, in which release rate requirements may vary for site development based on location within the watershed.

Implementation of these strategies should be evaluated in regards to their reduction benefit of peak runoff rates and high flow duration at the point, or points, of interest in the receiving channel for the overall watershed. The management strategy or hybrid of multiple strategies, providing the best results at the point of interest is selected for the optimal strategy for the watershed. This process essentially tailors the best fit for the watershed.

1.2 Research Objectives

The overall goal of this research is to investigate downstream runoff peak reductions resulting from application of alternative stormwater management strategies implemented throughout a branch of the Stroubles Creek watershed in Blacksburg, Virginia. Specific tasks necessary to achieve this goal are to:

1. Develop a hydrologic and hydraulic model representative of the subject watershed. Model development shall provide sub-basins within the watershed at a scale that provides the ability to represent areas treated by stormwater

management facilities. These areas are referred to as applicable areas within this report.

2. Review alternative stormwater management strategies and develop procedures to represent these strategies in the model within applicable areas.
3. Evaluate characteristics of runoff hydrographs resulting from each applicable area with each alternative management strategy. Routing will be subject to a range of design storms.
4. Analyze the resulting hydrographs at the watershed outfall generated from implementation of each alternative management strategy for all applicable areas within the watershed.

2 Literature Review and Background Information

Chapter 2 provides a review of the literature as it relates to traditional and alternative stormwater management strategies at the site scale and their cumulative impact at the watershed scale. The review is intended to:

- Identify management strategies that have been evaluated in the literature and those implemented at a site scale;
- evaluate the extent that stormwater management strategies have been evaluated at a watershed scale and
- identify the information needed for each strategy for model representation.

2.1 Stormwater Management Strategies at the Watershed Scale

Governing agencies typically enforce stormwater management regulations at the site development scale based on blanket regulatory strategies that require certain standards based on empirical research from other locations or from limited observations (McCaffery 2007). This practice has provided readily available guidance for stormwater management for development, but overlooks the fact that every watershed responds differently to rainfall and has unique issues related to runoff. Employing stormwater management, and therefore the installation of infrastructure, at the site development scale is a straightforward and cost-effective way for a locality to respond to address runoff issues resulting from urbanization. Goff and Gentry (2006) warn that policies which require uniform on-site detention should be used with caution since this may not address management needs for the overall watershed.

Emerson et al. (2004) questions if a peak-flow based strategy is meeting the intention of stormwater management regulations since total volume of runoff is usually significantly increased after development. The failures of the traditional peak reduction based strategy can also be noted by observation of degraded streams and flooding downstream of managed development. Prakash (2005) concludes that construction of detention basins is not likely to reduce stream bed degradation attributable to urbanization by an appreciable amount. This conclusion has led to the implementation of several alternative strategies, some of which are more closely examined later in this report. However, there is limited research presented in the literature that examines the effects of the alternative strategies at the watershed scale through a detailed modeling effort.

Better understanding of the shortcomings of traditional stormwater management at the site scale can be gained through the use of hydrologic/hydraulic modeling (Emerson et al. 2005; Moore et al. 2005) and the ability to utilize a Geographic Information System (GIS) to compile data needed for the analysis (Goff and Gentry 2006; Nehrke and Roesner 2004). A study performed by Emerson et al. (2005) is found to be the most relevant to this research. The study utilizes a calibrated hydrologic model to evaluate the effectiveness of flow reduction for a study watershed with an existing stormwater management system in Chester County, Pennsylvania. Simulations include modifying all detention facilities to replicate several alternative stormwater management strategies. Existing detention throughout the system is based on post-development peak rate limited to the predevelopment rate for the 2- through 100-year design storms. For a 24-hour duration, these design storms range from 3.25 – 7.54-inches, respectively. The study conducted simulated runoff for six measured storms ranging from 0.47 inches to 1.97 inches of rainfall. The discrepancy between design and storm simulation may explain some of the lack of effectiveness of detention found in this study since these facilities are typically designed for larger storm events. This research is further discussed in the following sub-sections.

There is significant literature emerging in regards to the optimal placement of Best Management Practices (BMPs) within a watershed by utilizing distributed hydrologic models and genetic algorithms (Perez-Pedini et al. 2005) or in conjunction with an optimization module (Carter et al. 2008; Lai et al. 2005; Riverson et al. 2004). Optimization is determined based on set goals and given predetermined locations, specified BMPs, and usually a cost function. It is determined that these studies do not explicitly provide insight into the effects of stormwater management within an overall watershed, but are rather more useful as support for local planners and engineers in the development of watershed plans (Lai et al. 2005). These tools could prove useful for maximizing public funds allocated for achieving Total Maximum Daily Load (TMDL) objectives (Carter et al. 2008), or otherwise maximizing the potential of public funds. Although placement optimization considers BMP options and their effect on the overall watershed, research is not focused on stormwater management regulations for land development, and therefore is not considered further in this review.

2.2 Traditional Stormwater Management

2.2.1 Maintaining Predevelopment Peak Runoff Rate

Traditional stormwater management regulations have required post development peak runoff rates to be no greater than the predevelopment rate for new development for specified design storms at the site scale (Pazwash and Boswell 2001). To comply with these regulations, detention ponds have become the predominant practice (Behera et al. 1999; Emerson et al. 2004; Fennessey et al. 2001; Pazwash and Boswell 2001).

Emerson et al. referenced three major shortcomings associated with the use of detention basins:

1. Although peak runoff rates are detained to below predevelopment rates, streambank erosion is increased with increased duration and volume of runoff (Malcom 1980; McCuen 1988; USEPA 2002).
2. The design of detention ponds are usually focused on specified return interval storms, such as the 2-year or 100-year storms, that account for only a small fraction of yearly precipitation.
3. Since detention ponds are designed on a site-by-site basis, their watershed-wide performance can not be assured (McCuen 1979).

Emerson et al. argues that storm water flows released from multiple sites eventually accumulate downstream, resulting in erosive potential in watershed channels. This accumulation phenomena is described by Urbonas et al. (2007) as the result of increases in post-urbanization runoff volumes and the summation of many small, flat and longer-duration hydrographs. Perez-Pedini et al. (2005) states that detention ponds can negatively impact the overall watershed when there is no systematic watershed-wide approach in the design of these detention facilities.

Emerson et al. (2005) used the U.S. Corps of Engineers Hydrologic Engineering Center's Hydrologic Modeling System (HEC-HMS) to perform a modeling effort on a 24 mi2 watershed (91 sub-basins with 82 detention ponds) in Chester County, Pennsylvania. Emerson concluded that on-site detention basins do not affect watershed-wide storm hydrographs resulting from frequent storm events. The study evaluated peak flow at the most downstream location in the receiving channel for each model run. One of the model runs compared a scenario with all detention ponds functioning as per a field survey (the existing condition) and compared those results to a run with all detention removed. Results showed that the detention ponds only reduced the peak runoff rate by an average of 0.3% for the six measured storms used for the study. Recall that these storms were small, more frequent events (0.47 to 1.97 inches). Considering that the 2-year design storm (± 3.2 inches) is typically the smallest required for adherence to stormwater management regulations (Nehrke and Roesner 2004), these results might have shown more peak reduction with larger storms.

Kaini et al. (2007) developed a model using the U.S. Department of Agriculture's (USDA) Soil and Water Assessment Tool (SWAT) and a genetic algorithm to determine the optimal pond area and volume for 159 sub-basins (each considered as a development site with a detention pond) within a 460 mi^2 watershed in Illinois. With the goal set to minimize flow at the outlet of the watershed, the results of the model were not surprising in that the optimal area for detention was essentially the upper limitation of 3.45 mi^2 (0.75% of the total watershed) set by the modeler. The results showed a reduction in maximum daily flow of 16.8%, compared to no detention for a 2-year simulation period using hydrologic data from the National Climate Data Center. The significance of the study, as related to this research, is mostly in regards to the potential to maximize site scale management at the watershed scale. However, findings inherently would require coordination of detention sizing throughout a watershed, which would be unrealistic to implement as a stormwater management strategy.

A study by Goff and Gentry (2006) seeks to find correlations between factors that affect the efficiency of detention for controlling peak flows throughout a watershed. A sensitivity analysis evaluated two watershed characteristics and several development characteristics. Each characteristic examined is summarized in Table 2.1.

Table 2.1 Summary of characteristics examined by Goff and Gentry (2006).

Watershed characteristic	
Shape	dendritic
	elongated
Slope	5% (mild) 15% (steep)

Development characteristic	
Size	20 acres
	80 acres
Intensity	low (SCS CN = 68)
	medium (SCS CN = 75)
	high (SCS CN = 92)
Sequence (downstream to upstream)*	100, 75, 50, and 25%
Sequence (upstream to downstream)*	25, 50, and 75%

*** Sequence was examined assuming medium intensity.**

Each development in the Goff and Gentry evaluation was managed with a detention pond designed to maintain the predevelopment runoff rate for the 10-year design storm within the synthetic watersheds. Rainfall was based on the SCS Type II 10-year design storm. Each development characteristic is evaluated for every combination of watershed shape and slope. The study evaluates flows at "1st" and "2nd" order stream junctions, concluding that detention:

- is most effective when development occurs in the upstream portion of the watershed;
- decreases in effectiveness as development increases;
- is not effective at maintaining predevelopment peak flow in the main channel, regardless of watershed and development characteristics and
- is less effective in watersheds with an elongated shape when compared to a more rounded shape.

2.2.2 Treatment component added to peak rate reduction

Spurred by Phase I and II of the Environmental Protection Agency's National Pollutant Discharge Elimination System (NPDES), more recent updates to stormwater management include a water quality component that typically requires treatment of the "first flush" (Pazwash and Boswell 2001). In Virginia, the Virginia Stormwater Management Regulations require that the first flush of runoff be captured and "treated" to remove pollutants. The first flush, or water quality volume, is defined as the first ½" to 1" of runoff from the impervious surface within the development (State of Virginia 1999). The water quality volume varies depending on the desired phosphorus (Virginia keystone pollutant) removal required and the removal efficiency credited to the Best Management Practice (BMP) selected for the subject site.

The first flush volume can be treated using a variety of BMPs that can extract the water quality volume from the runoff hydrograph altogether, such as with the use of an infiltration BMP, or release the volume over an extended period of time, such as with settling of pollutants via extended detention or filtering through a designed media. Note, however, that some proprietary filtering systems can be designed to pass flows without detention occurring. In these cases, regulatory requirements may be met in regards to pollutant removal, but no significant change occurs to the runoff hydrograph.

The addition of a treatment requirement can add an additional dynamic to the "managed" runoff hydrograph that previously was generated with the focus only on the reduction of the peak runoff rate. Without actually sizing to treat a water quality volume, Emerson et al. (2005) modified the outlet structures on ponds throughout the watershed to include an arbitrarily sized low-flow orifice of 4-inches to simulate a general low-flow orifice modification. This modification provides a simulation that could be considered to demonstrate the effects of extended detention throughout the study watershed. It also could demonstrate the potential benefit for a locality interested in retrofitting existing detention facilities.

For the six storms studied, the average peak flow reduction at the analysis point was 4%, when compared to the "no detention" simulation. Although the outlet modification described above demonstrates a trend towards a greater peak reduction than with detention alone, the following should be noted: extended detention is only one option available for treatment of flows and it is unlikely that all developments within a watershed would chose this BMP to achieve treatment compliance. Also, extended detention sizing criteria to treat the water quality volume would likely result in varying orifice sizes for each development, many less than the 4-inch orifices simulated by Emerson.

2.3 Alternative Stormwater Management

Some regulatory agencies that have recognized the shortcomings of the traditional stormwater management strategy have made conservative modifications to the post to pre-peak strategy. New Jersey requires post-development peak runoff rates for the 2-, 10- and 100-year storm events to be detained to 50, 75, and 80 percent of the predevelopment peak rate (New Jersey Department of Environmental Protection Division of Watershed Management 2004). Fennessey et al. (2001) evaluated five different stormwater ordinances at the site scale that all require peak runoff from a developed site to be reduced to, or below, the predevelopment peak but vary in regards to either the range of design storms for analysis, or post-runoff peak target (e.g. 75% of pre-peak runoff rate for specified design storms). Findings show that for large events (> 10 year), each ordinance achieved the objective at the site scale. However, a relevant finding demonstrates that management requirements ignore the smaller events (< 2-year) that often are the culprit of nuisance flooding in the overall watershed.

Others have conducted studies to evaluate innovative strategies and, in some cases, taken the initiative to develop and implement these strategies. Each of these strategies can be classified considering a watershed scale to have:

- blanket management requirements, in which all site developments within the watershed are subject to identical post-development peak rate and/or volume reduction requirements or

- stratified requirements, in which post-development peak rate and/or volume reduction requirements may vary for site development based on location within the watershed.

Also, although all strategies are intended to manage flows as they pass to the downstream receiving channels, each can either be classified as a watershed focused or site focused strategy. The difference being that a watershed focused strategy establishes site management requirements to maintain a baseline condition at a location downstream (a point of interest). These management requirements are established based on a watershed analysis. A site focused strategy is based on maintaining a specified condition, usually the predevelopment condition, at the outfall of the watershed. With site focused strategies, management requirements are not established from a watershed analysis. The most notable of these alternative management strategies, from a review of the literature, are described below.

2.3.1 Downstream peak focus *blanket* strategy

2.3.1.1 Unit Release Rates - Milwaukee

The Milwaukee Metropolitan Sewerage District's (MMSD) stormwater management regulations include an approach to demonstrate compliance to local stormwater regulations using specified maximum unit release rates (Milwaukee Metropolitan Sewerage District 2005). To comply, a designer shall demonstrate that post-development runoff from a site meets the following criteria:

- ✓ A release rate of 0.5 cfs/acre for the one percent probability event (100-year recurrence interval)
- ✓ A release rate of 0.15 cfs/acre for the fifty percent probability event (2-year recurrence interval)

The unit release rate (URR) strategy used by the MMSD are based on a comprehensive watershed analysis (Southeastern Wisconsin Regional Planning Commission 2007) and predominantly influenced by maintaining existing flow rates for a specified baseline and watershed timing (Milwaukee Metropolitan Sewerage District

2005). Although the MMSD encompasses six watersheds, the unit release rates described above were applied uniformly, a blanket requirement, "as the most efficient way to ensure existing flows and stages are maintained in all downstream areas."

The URR appears promising for mitigating for increases in flows and stages throughout a watershed. However, some localities are reluctant in adopting this approach. The predominant obstacles for implementation of a URR approach is collecting the extensive data and performing the modeling needed to determine defensible release rates, as recognized by the Waukesha County Stormwater Advisory Committee (2004). This approach also does not appear to explicitly address the first two of Emerson's shortcomings previously mentioned in Section 2.2.1; those shortcomings are summarized as the increased duration and volume of runoff and the management for the smaller, more frequent storm events. Since a watershed analysis was performed to generate release rates, it may be considered that Emerson's third shortcoming has been considered.

2.3.1.2 Standardized Release Rate – Overland Park

McEnroe and Heatherman (2006) conducted a modeling effort that sought to simplify management requirements to reduce administrative workload for the City of Overland Park, Kansas. The goal was to develop a detention design requirement that would provide assurance of no increase in flooding at downstream problem sites (defined by Overland Park as no increase in the 100-year peak runoff rate) and no nuisance flooding directly downstream of the development site (defined as no increase in the 10-year peak runoff rate).

Routing iterations were performed for a hypothetical 10-acre development site with a typical detention design and determined that the peak from a release rate of 3.0 cfs per acre for the 100-year storm event would not surpass the predevelopment 10-year peak discharge from a development site. To address downstream flooding, a hypothetical 200-acre drainage area consisting of twenty 10-acre sub-basins situated as shown in Figure 2.1 was utilized to evaluate several standardized release rates applied to all sub-basins. Since Overland Park detention policy had generally considered detention in the lower one-third of a watershed less effective, iterations were also run that omitted any management from the lower one-third of the watershed. The study found that the peak from a release rate of 3.0 cfs per acre for all developed parcels (sub-basins) did not surpass either the 10- or 100-year predevelopment peak runoff rate at the downstream reach.

In the case with detention in only the upper two-thirds of the watershed, the goal for detaining to the 10-year predevelopment peak discharge baseline was not being met. Provided results demonstrate for both the 10- and 100-year peak runoff rates an approximate 20% increase when the lower one-third of the basins do not have detention, compared to all basins having management. This could indicate that although the peaks from the basins in the lower one third may pass prior to the overall watershed peak, the receding limbs of their runoff hydrographs still contribute significant enough flows to the overall peaks. It should be noted that the study also found there to be negligible increase in peak runoff when development was confined to the lower one-third, indicating the timing combination of a quicker peak down stream and the slower timing of the predevelopment peak in the upper two-thirds, offsets the increase in runoff in regards to the contributions to the overall watershed peak.

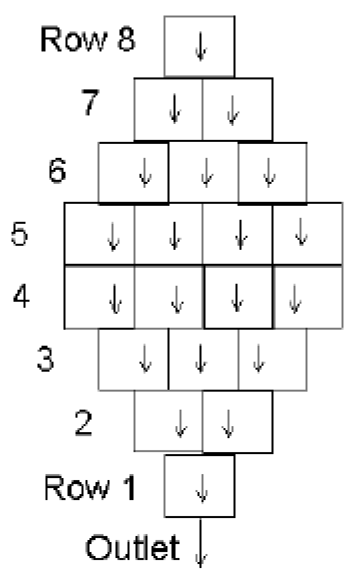


Figure 2.1 Hypothetical 200-acre watershed. (McEnroe and Heatherman 2006)

In summary, the study concludes that a standardized release rate of 3.0 cfs per acre for the 100-year storm with new development met the goals for no increase in flooding at downstream problem sites and no nuisance flooding directly downstream of the development site. As with other site-based peak-focused strategies, the three major shortcomings listed by Emerson et al. (2005) are not addressed with this type of strategy. Those shortcomings are summarized as the increased duration and volume of runoff, the management for the smaller, more frequent storm events and the assurance of watershed-wide performance. Considering the assumptions associated with the hypothetical watershed there may not be merit for utilizing these findings for establishment of stormwater management regulations given the variability in a natural system and the lack of consideration in this study on smaller storm events. Therefore, this strategy is deemed not worthy to be considered in the analysis portion of this research.

2.3.2 Downstream peak focus *stratified* strategy

2.3.2.1 Stratified release rate - development

Recognizing that the location of stormwater management within a watershed can affect the peak of the overall watershed, the strategy of stratifying detention, mostly based on travel time to an analysis point, has been introduced as a management strategy. Emerson et al. (2005) evaluates this strategy at a watershed scale in a much generalized simulation by removing detention for the lower half of the sub-sheds (based on travel distance to the analysis point) within the study watershed. The strategy of not detaining flows in the lower half of the basin is based on the assumption that the majority of these flows will pass a critical location before the upper flows reach a downstream critical location. The sub-basins with detention in the upper half of the watershed were simulated as the previously described low-flow (extended detention) configuration. Results for the six storm events simulated show only a 0.8% reduction in flows when compared to the previously described no detention option.

The arbitrary split of one-half of sub-basins to have no detention, and extended detention used in the upper reaches, may not provide an adequate evaluation of stratification of detention. But similar to the previously mentioned study by McEnroe and Heatherman (2006), where no management is evaluated in the lower one-third of the watershed, some insight can be gained in regards to the effects on the peak runoff rates for the overall watershed when runoff release requirements are stratified throughout the watershed.

A more detailed designation for stratification that specifies varying release rates throughout the watershed is presented by Shamsi (1996). Shamsi utilized a lumped parameter hydrologic model to generate runoff hydrograph contributions at specified downstream reaches for each individual upstream sub-basin within the 8.4 mi^2 Bull Run watershed in Union County, Pennsylvania. The release rate (% of predevelopment runoff) for a specific sub-basin is then calculated for each specified downstream reach as:

$$R_{i\,j} = \left(\frac{Q_{ij}}{Q_i}\right) \times 100 \qquad (2\text{-}1)$$

Where: R_{ij} *= release rate of sub-basin i at downstream reach j (% of predevelopment peak runoff rate)*

Q_{ij} *= predevelopment discharge contribution of sub-basin i to the peak discharge of a downstream reach j (cfs)*

Q_i *= predevelopment peak discharge of sub-basin i (cfs)*

The predevelopment discharge contribution Q_{ij}, at a point of interest can be considered as the flows from a sub-basin *i* runoff hydrograph that are contributing to the combined hydrograph at a downstream reach *j*. The combined hydrograph at the downstream reach is the result of the summation of upstream hydrographs. Contributing flows from the upstream sub-basin are heavily influenced by timing. This description is graphically illustrated in Figure 2.2 in the next section. Shamsi describes a method for this evaluation using a peak flow presentation table (PFPT). The table provides flow rates at time increments that represent the runoff from each sub-basin contributing to a point of interest. Runoff is lagged by the travel time from the sub-basin outlet *i* to the point of interest *j*. The summation of the flows at each time step generates the hydrograph at the point of interest, the contribution from each sub-basin at the time of peak discharge at the point of interest is Q_{ij} for sub-basin *i*.

Based on the tabulated results for each sub-basin, conservative rules are used that require the lowest release rate for each sub-basin (for all specified downstream reaches) to be the designated release rate. Once the values are determined, the following concepts lead to generalized adjustments to the calculated rates:

- A calculated release rate of 0% would be difficult to achieve and would most likely prevent the development of a site. In the case where the calculated value is 0%, Shamsi recommends an even compromise of 50% release rate for these sub-basins.
- A calculated release rate below 50% indicates a sub-basin's peak occurs significantly prior to the watershed peak. Maintaining a 50% release rate could result in large detention facilities and therefore the author recommends these sub-basins be allowed to release 100% of the predevelopment flows so that high runoff rates from these sub-basins can pass prior to the watershed peak. (For contrasting comparison, recall McEnroe and Heatherman (2006) found a 20% increase in peak runoff rate for the watershed when detention was removed from the lower one-third of the watershed.)
- The calculated release rates of the uppermost basins tend to be significantly different from those sub-basins immediately downstream. Shamsi states these upper basins should be assigned the release rate of the next downstream basin to address the concern of greatly differing release rates from those assigned to adjacent properties.

Adjustments to finalize release rates based on the concepts above and rounding values to the nearest tens provide more generalized values. Adjacent sub-basins with the same finalized release rates are then grouped together to produce a release rate map. It should be noted that release rates presented by Shamsi correspond to the 100-year design storm, which is said to produce the smallest (most stringent) release rates. Release rates are then implemented by identifying the applicable release rate from the map and applying the rate as:

$$Q_{pi} = \left(\frac{R_i}{100}\right) \times Q_i \qquad (2\text{-}2)$$

Where: Q_{pi} *= allowable post-development peak discharge from sub-basin i*

R_i *= release rate of sub-basin I from release rate map*

The presentation by Shamsi provides guidance towards the generation of a stratified strategy to stormwater management, but no modeling analysis was performed to evaluate management in place that meets the prescribed release rates. Distributed modeling efforts, with detention facilities designed to comply with the assigned release rates, was not found in the literature.

2.3.2.2 Stratified release rate - Implementation

An entity that has performed the modeling efforts towards implementation of a stratified release rate strategy is the Lehigh Valley Planning Commission (Lehigh Valley Planning Commission 2000). In contrast with blanket strategies, the Lehigh Valley Planning Commission (LVPC) provides mapping that guides stratified peak reduction, or no reduction, requirements for development throughout the watershed.

The goal of the LVPC approach is to prevent drainage problem areas from getting worse as a watershed develops. The philosophy to meet this goal is based on stormwater requirements at a site scale to be based on a watershed analysis that assures no increase in peak flow rates at specified problem areas. Release rates are established throughout a watershed based on a watershed-wide analysis that is predominantly based on location in the watershed in relation to a point of interest, hence travel time. Figure 2.2 demonstrates the LVPC strategy by illustrating the following:

- Runoff from sub areas 1 and 5 *do not* contribute to the peak runoff of the overall watershed. Therefore, no control is a valid option.
- Runoff from sub area 4 *does c*ontribute to the peak runoff of the overall watershed. In this case, sub area 4 would be required to detain the post-

development peak runoff rate to the equivalent of the pre-runoff rate at the time of occurrence of the overall watershed's peak rate.

- Runoff from sub area 2 *does c*ontribute to the peak runoff of the overall watershed. The appropriate strategy for sub area 2 would be to provide detention to assure that any post-development rise in the hydrograph would not increase the overall peak.
- Runoff from sub area 3 *does c*ontribute to the peak runoff of the overall watershed. Since 100% of the areas peak contributes to the overall runoff so the requirement would be to detain to the predevelopment runoff rate.

In summary, identical to Shamsi's equation, the following is concluded as the prescribed release rate for a specific sub area:

$$Release\ rate\ \ (\%\ of\ \ pre-peak) = \frac{Subarea\ contribution\ to\ location\ of\ Interest}{Subarea\ \ peak\ \ flow} \qquad (2\text{-}3)$$

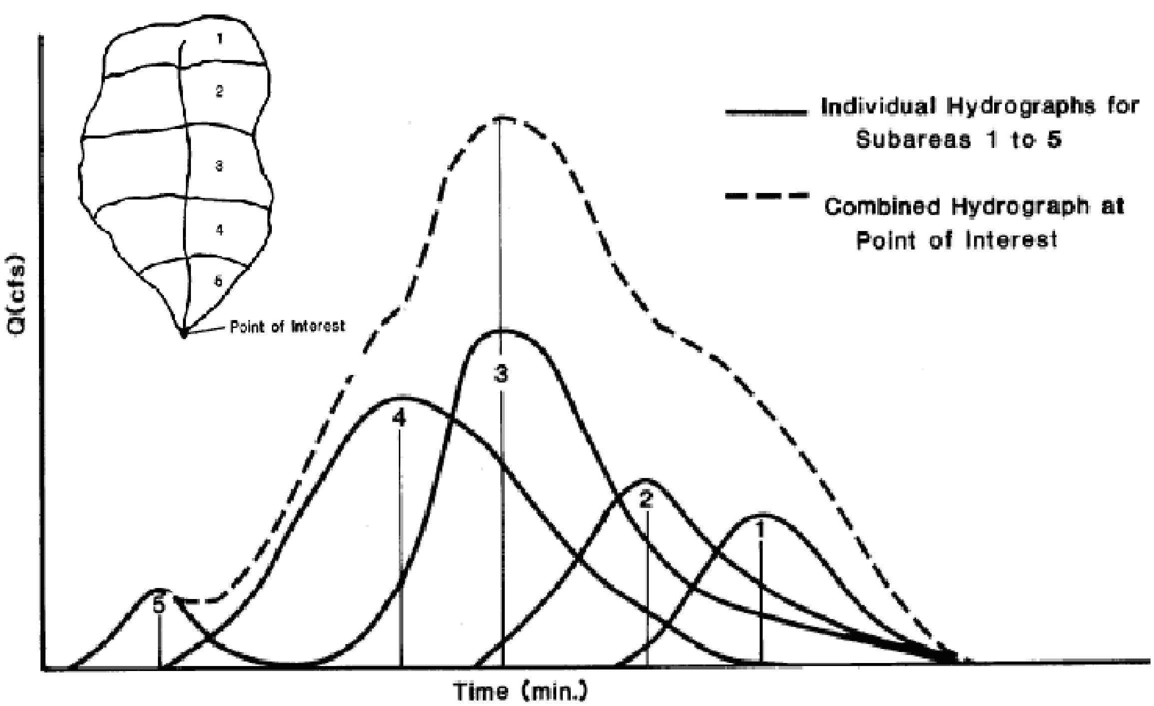


Figure 2.2 Hydrograph analysis example for determining release rate requirements (Lehigh Valley Planning Commission 2000).

The result of the modeling effort and strategy described above establish two basic districts in the Little Lehigh Creek Watershed. The first includes areas that have early peaks and require no detention, although adequate downstream channel must be demonstrated. The second district must provide management that reduces peak runoff rates to a specified percent of the predevelopment peak for the 10-, 25-, and 100-year post-development runoff (varying from 50-100% based on peak timing). It should be noted that watershed analysis determined that all sites in this district shall reduces peak runoff rates to 30% of the pre-development peak runoff rate for the 2-year storm.

2.3.3 Volume focus strategy

An increasingly popular strategy to stormwater management involves subtracting a portion, if not all, of the volume from a site's runoff hydrograph, the former being more realistic in most cases. Reduction of volumes can be achieved by reducing impervious cover and routing flows over pervious surfaces along with the use of a variety of BMPs, such as bioretention or infiltration practices. As a stormwater management practice, this approach is most commonly identified as Low Impact Development (LID), originally presented by Prince George's County, Maryland.

Emerson et al. (2004; 2005) provides simulation of volume reduction by extracting the first 0.5, 1.0, and 2.0-inches of runoff draining to each detention basin in the study watershed. For the six storms studied, when compared to the no detention option, the average peak flow reduction at the analysis point was 9.2, 12, and 13%, respectively. Emerson made no design consideration to explicitly demonstrate reduction of the post-peak runoff rate to the predevelopment rate for the modeled detention facilities.

2.3.3.1 Low Impact Development

Prince George County (PGC), Maryland, initially developed and implemented the Low Impact Development (LID) approach to manage stormwater (NAHB Research Center 2003). LID is intended to guide design that maintains or restores a development to the predevelopment hydrologic and hydraulic conditions. The optimal result is that the post-development runoff hydrograph matches the predevelopment hydrograph by using a combination of retention (infiltration) and detention along with design components that maintain the time of concentration. Since PGC's implementation of LID, many municipalities and states are revising ordinances and regulations in an effort to preserve predevelopment hydrology (McCaffery 2007). Numerous sources now exist that provide site design guidance to strive to maintain the predevelopment hydrologic conditions with the use of foot-printing, disconnected impervious areas, pervious surfaces and integrated management practices (IMPs).

A challenge with LID is the need for computational procedures that can be readily accepted by regulators. To address this issue in PGC, a methodology is provided for hydrologic analysis and computational procedures based on the Soil Conservation Service (SCS) TR-55 hydrologic model (Prince George's County 1999). These computation procedures initially guide the designer towards minimizing the runoff curve number (CN), to develop an "LID" CN, and maintain time of concentration through design strategies. The procedures then offer a series of charts that provide storage volume requirements according to the rainfall depth (SCS Type II, 24-hour storm) and the predevelopment CN and LID CN known. These charts are identified as:

- Chart Series A: Storage Volume Required to Maintain the Predevelopment Runoff Volume Using Retention Storage (example, Appendix A, Page 7-1).
- Chart Series B: Storage Volume Required to Maintain the Predevelopment Peak Runoff Rate Using 100% Retention (example, Appendix A, Page 7-2).
- Chart Series C: Storage Volume Required to Maintain the Predevelopment Peak Runoff Rate Using 100% Detention (example, Appendix A, Page 7-3).

The storage volume designations by the charts are in hundredths of an inch over the entire area of the site. The area needed for retention is determined based on a specified depth for the IMPs (typically 6-inches for bioretention, rain-gardens, etc.). These charts are generated based on the assumption that IMPs are evenly distributed throughout a site, the site is relatively homogeneous, and, in the case of Chart Series B &

C, the predevelopment time of concentration is maintained so that runoff rates are independent of time of concentration.

After determining retention needed to maintain predevelopment *volume*, the procedures provide guidance for utilizing Chart Series B and/or C to determine the volume of retention and/or detention needed to maintain the predevelopment *peak* runoff rate. These charts are based on the relationship of peak outflow to peak inflow discharge (q_o/q_i) and storage volume to runoff volume (V_s/Vr). For detention (Chart series C), PGC uses this relationship demonstrated in Figure 6-1 (see Figure 2.3) of the TR-55 Manual (SCS 1986). In general, Chart Series C serves as an acceptable approximation for estimating detention storage for IMPs dispersed throughout an LID site design where retention is not practical and soils are not amenable to infiltration.

Whereas design storms required for stormwater management design are typically the 2-year (channel protection) and 10-year (flooding considerations) events, an alternative design storm is presented by PGC for LID. The LID design storm is to be whichever of the following results in the greater rainfall:

- Rainfall at which direct runoff begins from a site that assumes a CN based on woods in good condition, with the existing hydrologic soils group (HSG). Equation 2.2 from TR-55 (initial abstraction) is used to determine the rainfall needed to initiate runoff, with a modifying factor; or
- the 1-year, 24-hour storm event.

The PGC procedures provide the ability to quantify runoff reduction based on site design through reduction in the runoff curve number and also provide the ability to account for runoff reduction through the use of IMPs. Quantifying the effects of these practices could prove overly complex if traditional routing procedures were implemented. The PGC analysis and procedures are intended to provide reasonable, defendable criteria for reviewing agencies to have a comfort level in approving an LID stormwater management plan.

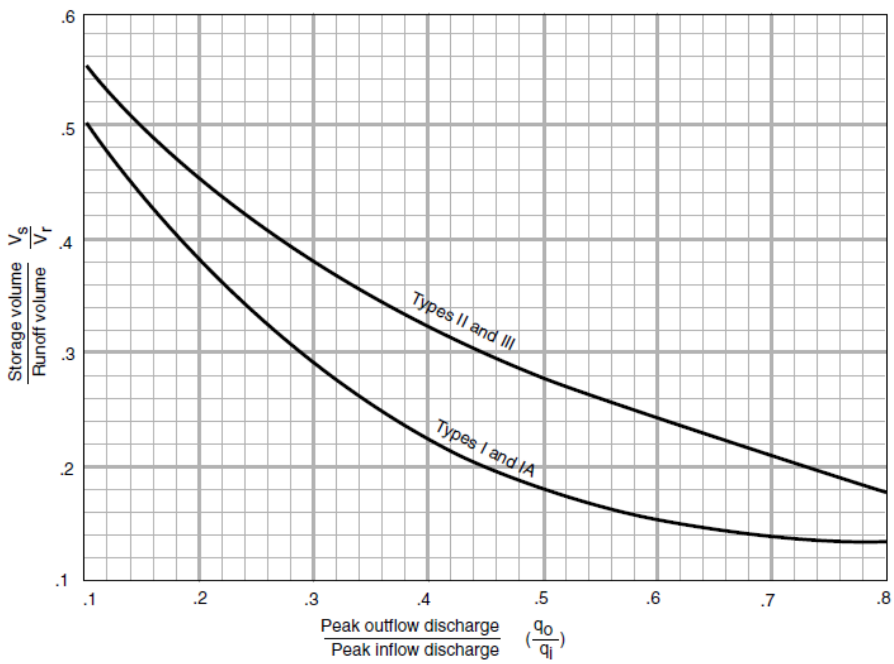


Figure 2.3 Approximate detention basin routing for rainfall Types I, IA, II and III (SCS, 1986).

Although LID appears to address Emerson's three detention shortcomings, limitations exist that potentially impede the successful use of LID strategies, some regulatory and some physical. In Virginia, design would also require analysis under the 10-year storm event (State of Virginia 2001) to demonstrate the predevelopment peak runoff rate is maintained, causing a conflict with the use of the PGC design storm. Considering the integrated nature of stormwater management throughout an LID site, an analysis for the 10-year storm event would most likely require a complex routing analysis. This presents challenges for designers who would be tasked to replicate the design with modeling software in a manner that would be acceptable, and defendable to the reviewing jurisdiction or agency. With this, modeling of LID and its site components have become an issue of much discussion and focus in more recent literature. The scope of the research presented here maintains focus on management strategies at the watershed scale and does not attempt to provide micro-modeling guidance at the site level for LID.

Physical challenges in successful LID implementation exist where soils are not amenable to infiltration, karst conditions are present, or for redevelopment in urbanized areas (consequently potentially encouraging sprawl). However, it should be noted that since flows are retained (delayed) significantly, practices such as bio-retention, with under-drains, can be utilized where poorly drained soils exist. The assumptions for the Chart Series described above cause concern for error in assurance of maintaining the predevelopment time of concentration and evenly distributed retention/detention facilities throughout a site. While the concept behind LID appears to provide the optimal solution for on-site stormwater management, LID is best suited for new, suburban residential development (NAHB Research Center 2003).

2.3.3.2 Full Spectrum Detention

The Urban Drainage & Flood Control District (UDFCD), which serves 40 Colorado Counties, Cities, and Towns, has adopted a design strategy called Full Spectrum Detention (FSD). The unique component to FSD is the extended detention of the excess urban runoff volume (EURV) which is defined as the difference between the post- and pre-development runoff *volume*. The EURV value is established as inches per acre utilizing a full range of design storms. Methodology for establishing this value is described in Section 3.3.4.2. Urbonas and Wulliman (2007) evaluated the FSD design methodology intended to offer more “robust” controls of the more frequent, smaller runoff events. The study found that when the EURV plus 10% is captured and released over 72-hours, the runoff volume remaining after the EURV approximates the predevelopment volume.

The Urbonas and Wulliman evaluation modeled the FSD protocol within a 3.1 square mile watershed with gauged data and then a 9.3 square mile (5,000 acre) synthetic watershed to evaluate at a larger scale. Results found the protocol to maintain the predevelopment peak runoff rate at the outfall of the watershed for a full range of design storms, when implemented in all sub-basins within the watershed. In each case, model simulations for sub-watersheds were subject to detention that met the following runoff requirements:

- Detention and release of the EURV over a 72-hour drawdown period.
- Control of the 100-year peak runoff rate to the predevelopment peak (in the case of the UDFCD areas, predetermined release rates based on 2% imperviousness and SCS soils designation)

Localities within the UDFCD are beginning to adopt the FSD design protocols adopted by the district. The authors of the research note that findings are most applicable to areas with similar rainfall and that the prescribed controls provide the desired watershed-wide goals only when implemented within 100% of the watershed. In addition to the author's limitations, the large detention volumes required for a 72-hour drawdown should also be considered. In comparison, the Virginia Stormwater Management Handbook (State of Virginia 1999) requires a 30-hour drawdown, with a water quality focus for extended detention, stating that this time exceeds probable settling time for pollutant removal.

2.3.3.3 Volumetric Design Procedure

The volumetric design procedure (VDP) is an alternative approach allowed by Milwaukee Metropolitan Sewerage District when the URR (refer to section 2.3.1.1) is not practical (e.g. significant off-site flows through the subject site). This approach recognizes that design controls can rarely provide for post-development runoff volume to be reduced to the predevelopment runoff volume. Recognizing this limitation, VDP requires that post-development runoff volumes shall be limited to the predevelopment runoff volumes during a critical time period for both the 100-year and 2-year events. The critical time period is defined as the amount of time from the peak of the design rainfall to the peak of the runoff hydrograph resulting from a subject watershed at some critical point (see Figure 2.4).

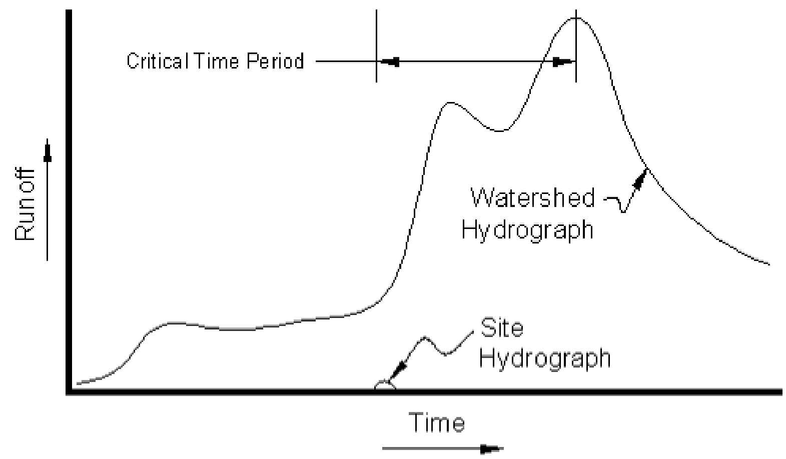


Figure 2.4 Visual depiction of the critical time period. Note that SCS hydrology based on a 24-hour storm is used for site and watershed hydrology, resulting in a critical storm period that could be consistent for any site within the watershed. (Milwaukee Metropolitan Sewerage District)

This approach requires the determination of critical locations and a comprehensive watershed modeling analysis for determining the critical time period. A critical time period is therefore assigned within the drainage basin for each critical area. The MMSD watershed model provides critical time periods for specified basins within the jurisdictions watersheds. As with the MMSD URR strategy, some localities are reluctant in adopting this methodology due to the time and expense needed to generate the information for implementation. No watershed-wide modeling effort to evaluate the effects of this strategy at a watershed scale is presented in the literature.

3 Methodology: Model Development & Strategy Representation

Chapter 3 presents the methods used to achieve the first two objectives described in Section 1.2. Methodology objectives to support the overall objectives are described as:

1. Development of a hydrologic and hydraulic model representative of the subject watershed.
2. Development of procedures to represent alternative management strategies in the model within applicable areas.
3. Description and discussion regarding the blanket and tailored strategies to be modeled and compared.

The process and techniques used to develop the urban watershed model and evaluate the watershed runoff response to alternative strategies is discussed. Section 3.1 discusses the creation of the hydrologic/hydraulic model in Bentley's SewerGEMS software, including data collection and generation. This section also discusses model calibration. Section 3.2 presents the processes used to represent alternative strategies within the model, including the determination of the critical storm duration for the Volumetric Design Procedure. Section 3.3 discusses the scenarios to be modeled.

3.1 Watershed Model Development

3.1.1 Subject Watershed

The watershed selected for study is a branch of the headwaters of Stroubles Creek in Blacksburg, Virginia (Appendix B, Page 7-4). The watershed encompasses 381-acres and is nearly completely built-out consisting of residential development of varied density, a 10-acre park, a 10-acre cemetery, Blacksburg High School and Harding Elementary School. The selection of an urbanized watershed was based on available detailed data and the ability to analyze the effects of alternative management strategies in a practical setting, given that only portions of the watershed are subjected to stormwater management. The watershed contains nine existing stormwater management facilities, which is a reasonable number to provide design for each for representation of each alternative strategy within a model. Since the subject watershed is already predominantly built-out, results are also intended to provide insight as to the optimal strategy to implement for retrofit of facilities serving applicable areas. Recall from the Problem Statement (Section 1.1): "Protection of downstream channels and reduction in flooding can potentially be improved by evaluating alternative stormwater management strategies at a watershed scale." The selection of the subject watershed offers a scale, number of sub-sheds and management facilities that allows for each specific alternative strategy to be sufficiently represented and analyzed.

The watershed has been segregated into 35 sub-basins (Appendix B, Page 7-5). These sub-basins within the study watershed were delineated at a scale that considered areas draining to stormwater management facilities, areas with homogeneous land cover and areas that drained to a location upstream of significant confluence in the storm sewer system. The average sub-basin is 11.2 acres, with a maximum of 30.3 acres (homogeneous with no storm sewer infrastructure) and a minimum size of 1.1 (homogeneous apartment complex). The wide range is representative of the variations that can occur within an urban watershed. A distribution of sub-watershed size is provided in Figure 3.1.

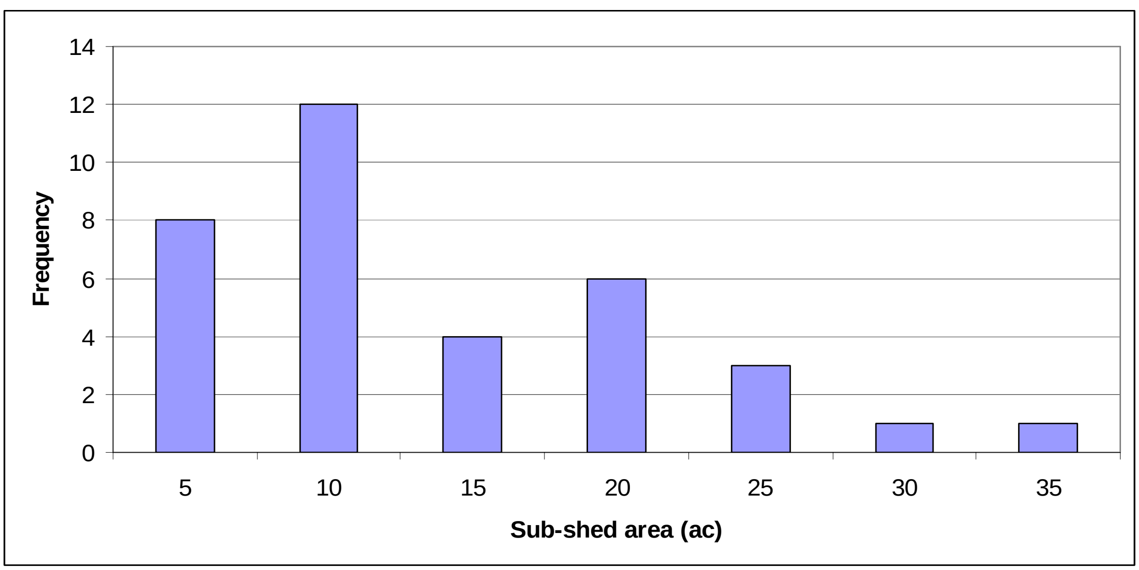


Figure 3.1 Histogram illustrating the distribution of sub-watershed size. For example, 8 sub-sheds have areas between 0-5 acres and 1 watershed is between 30-35 acres.

The analysis point (lowest point in the watersheds main channel) is a 4.5'H x 6.0'W box culvert beneath Progress Street just downstream of a channel adjacent to the Progress Street Fire Station. Downstream of the analysis point, the creek is piped beneath downtown Blacksburg and the Virginia Tech campus, areas historically subjected to flooding due to inadequate capacity for flows. Damage to structures has occurred in both downtown Blacksburg and on the Virginia Tech campus during large storm events. Several regional stormwater facilities have been constructed within the subject watershed based on previous studies in an attempt to reduce the impact of high flows. Streambank erosion also is an issue in the rural areas further downstream. The piping of the stream beneath downtown and campus ultimately day-lights just upstream from the Virginia Tech Duck Pond (see Appendix B, Page 7-6).

An urbanized watershed was deemed critical for evaluation of alternative strategies towards “tailored” stormwater management. Whereas new development is now subject to stormwater regulations, older development (prior to the early 1980s in Blacksburg) was not subject to regulations and therefore many developments in urbanized watersheds have no management in place. In developments where traditional management does exist, it did not consider the effects of the post- to pre-peak runoff rate strategy on the overall urbanized watershed (i.e. timing of flow release, summation of flows within conveyance systems). Furthermore, since localities are taking a more holistic environmental approach and encouraging infill development and redevelopment in more recent years, it is important to understand which stormwater management strategy may be the most beneficial in these areas to improving conditions at a critical location.

3.1.2 Watershed Data

Stormwater data compilation and modeling efforts within the Town of Blacksburg (TOB) Engineering and GIS Department have been compiled in a piecemeal fashion in the past and have not been well documented. With this, there was little confidence in the data available and previous modeling efforts had been lost with computer and software updates over the years. Fortunately, the timing of this study corresponded with the initial phase of the TOB's efforts towards stormwater infrastructure inventory and development of a town-wide stormwater model. The subject watershed presented in this report also serves as the pilot watershed for the Town. Data for the pilot watershed was collected in the summer and fall of 2008 by the TOB and the Center for Geospatial Information Technology (CGIT) at Virginia Tech. Data collection involved field data collection and review of record drawing (as-built) information. All data was stored in geo-database format tailored for import into modeling software. More detailed discussion regarding data collection, compilation and model development is described in the next section.

Initial discussion between the TOB and CGIT focused on the selection of the runoff method to be used with the model, identifying the intended uses of the model and determining the level of accuracy needed for data collection. SewerGEMS modeling software was selected since it was already being used by the Town, its ability to integrate with Geographical Information Systems (GIS) software and its ability to export data to the Environmental Protection Agency's SWMM modeling software, a commonly used and publicly available modeling software. The SewerGEMS implicit engine, which uses a four-point implicit finite difference solver to solve the St. Venant equations, was selected for hydraulic calculations within the model. Detailed specifications for calculation options are provided in Appendix C, Page 7-7.

Selection of a runoff method was dependent on the Town's desire to use a method familiar to the local development community and the need for input data to be understandable and defendable to future staff and other local professionals. SewerGEMS offers several runoff methods as options, including Unit Hydrograph, EPA SWMM and Modified Rational. With this, SWMM runoff was not considered due to additional input parameters required such as characteristic width and storage depression depths, for example. These types of parameters are not commonly used in the development community and compilation of this type of data would inherently be subjective and result in some uncertainty. The Modified Rational method was also not considered since this method is most frequently recommended for drainage areas less than 20 acres. The Soils Conservation Service (SCS), now the Natural Resources Conservation Service, Unit

Hydrograph Runoff Method (U.S. Department of Agriculture 1986), from this point on referred to as SCS hydrology, was deemed as the most appropriate method for the TOB needs. Since the SCS Unit Hydrograph Runoff Method is typically used in stormwater management design for development, it is also considered as an appropriate runoff method for the purposes of this research.

3.1.2.1 Hydrology - Curve Number

The primary parameter for the SCS Unit Hydrograph Runoff Method is the SCS curve number (SCS CN). The CN is an empirical parameter that predicts direct runoff or infiltration from excess rainfall. The SCS CN method is described in the National Resource Conservation Service's (NRCS) Technical Release Number 55 (TR-55). The method involves designation of a CN using tables provided in Chapter 2 of TR-55 based on the hydrologic soils group (HSG) and the land cover condition of the subject drainage basin. A secondary characteristic that can further refine a CN value is the percent impervious connected or disconnected. This secondary characteristic becomes more relevant for generation of a CN using LID methodology, described in more detail in Section 2.1. Curve numbers are unit-less and range from 30 – 98; with higher numbers producing more runoff (i.e. roads and rooftops would have a CN of 98, independent of soils group).

CN data for the pilot watershed was created by CGIT with the development of a GIS vector based dataset. In general, datasets were created by intersecting spatial soils (HSG) data and land cover data and assigning a CN based on the TR-55 CN tables. CGIT created three separate datasets, each utilizing a varying levels of scale for land cover data (see *CGIT's CN Development Report* in Appendix D). The smallest scale is identified as "detailed" land cover. As noted in the CGIT report, the detailed land cover data was generated utilizing aerial photography, existing road edge and buildings GIS data and student workers to generate highly detailed data, at a spatial resolution of sidewalks and driveways. It was determined that this detailed land cover dataset was the most desirable to use for the purpose of this research since sub-basins are delineated to a relatively small scale within the overall watershed (recall that some sub-sheds are as small as 1.06 acres).

3.1.2.2 Hydrology - Time of Concentration

A second critical parameter required for the SCS runoff method is the time of concentration (t_C). T_C is described in TR-55 as: "the time it takes for runoff to travel to a point of interest from the hydraulically most distant point." The t_C can be considered the hydrologic response time of a drainage area. The t_C is usually considered the sum of the travel times on each segment of the principal flow path. These segments are defined as sheet flow (occurring in the upper reaches of the flow path), concentrated or shallow flow (generated as sheet flow concentrates in rills and swales) and channel flow (pipes and streams). A variety of methods are available for determining travel time (T_T) in flow paths segments, both for sheet flow and concentrated or channel flow. For the purpose of this research, the Seelye Equation (Equation 3-1) was used for sheet flow segments and the Kirpich Equation (Equation 3-2) for concentrated and channel flow segments.

$$T_T = 0.225 L^{0.42} S^{-0.19} C^{-1.0} \qquad (3\text{-}1)$$

$Where: T = Travel\ time$, min
$L = Length\ of\ \ flow\ path\ segment\ (or\ strip),\ ft$
$S = Slope,\ ft/ft$
$C = Rational\ "C"\ value\ (impervious = 0.9,\ pervious = 0.3)$

$$T_T = 0.0078 L^{0.77} S^{-0.385} \qquad (3\text{-}2)$$

$Where: T = Travel\ time$, min
$L = Length\ of\ \ flow\ path\ segment,\ ft$
$S = Slope,\ ft/ft$

Note that the Seelye Equation is a function of length, slope and land cover and the Kirpich is a function of length and slope. To generate times of concentration for each sub-basin in the subject watershed, flow paths were generated as a GIS shapefile and attributed with the required information. This process began with the TOB digitizing flow paths within each sub-basin in the watershed utilizing aerial photography, topographic data and storm sewer information. Several flows paths were generated within each basin to assure the longest flow path was utilized for time of concentration calculations. Flow path data was digitized in segments consisting of overland flow and concentrated/channel flow segments. Each segment was attributed with an associated sub-basin ID, whether it was grassed or paved and its length. The “grassed or paved” attribute was used to assign a generalized “C” variable for Equation 3-1 for overland flow. Once flow paths were generated, topographic data was used to assign up- and down-stream elevations to each flow segment, which along with the length information, allowed for the generation of a slope attribute for each segment.

3.1.2.3 Hydrology - Rainfall

With the characteristic parameters of the sub-basins obtained, the remaining information needed for the hydrologic component of the model is rainfall. Similar to reasons in the selection of a runoff method, SCS synthetic Type II, 24-hour design storms are evaluated for the purpose of this research, as opposed to measured rainfall data. Additional reasons for using SCS design storms are the lack of a wide range of measured storm events and the consistency with the methodology used to represent some of the alternative strategies, as is further described in Section 3.2. Precipitation depths used for the SCS Type II, 24-hour rainfall distributions are obtained from the National Oceanic and Atmospheric Administration's (NOAA) Precipitation-Frequency Atlas of the United States (Appendix E). Table 3.1 provides the depth used for the range of typical 24-hour design storms used with stormwater management design in Blacksburg, Virginia.

Table 3.1 Precipitation depths from NOAA for specified 24-hour design storms.

Recurrence Interval (yr)	Precipitation Depth (in)
1	2.31
2	2.80
5	3.56
10	4.18
25	5.09
100	6.66

It was found while routing the range of storms described in Table 3.1, that overflows occurred throughout the watershed with the 25- and 100-year storm events.

This is to be expected and can be explained by the fact that stormwater conveyances are typically designed to only pass up to a 10-year storm. Likewise, stormwater management facilities have typically been designed only to detain flows up to the 10-year storm with larger storms only being provided "safe" passage. It is typically not feasible to detain larger storm events. With these larger events, safe passage usually is provided with a weir and description of flow path to demonstrate structures would not be encroached with larger storm events due to the subject development. Since stormwater management for development is typically focused on smaller, more frequent storm events, for the purpose of this research, the 1-, 2-, 10- and 25-year storm events will be used for comparison of the alternative management strategies.

3.1.2.4 Stormwater Conveyance System

Stormwater conveyance system infrastructure data was collected as part of the collaborative effort between TOB and CGIT. Data was collected for the purpose of infrastructure management and to provide the data needed for detailed modeling efforts. CGIT graduate assistants and TOB interns collected data for the subject watershed in the summer of 2008. Global Positioning System (GPS) equipment was utilized for horizontal positioning of structures. Record drawing and digital aerial photography were used to assist in locating structures in the field. In addition, all other relevant information for modeling purposes was collected, including: pipe size, geometry and type; and channel cross-sections and type. Town topographic information was utilized to assign top elevations for manhole and catch basin structures. With top elevations assigned, inverts could be assigned based on field measurements from the tops of structures to the pipe(s) invert at the structure. The horizontal and vertical data collected was used to assign pipe slopes. Similarly, topographical information, along with channel length was used to generate channel slopes. All information was stored in GIS shapefile format with the corresponding attributes, and subsequently a SewerGEMS geo-database. A summary of data collected for the subject watershed and imported into the model is summarized in Table 3.2. Detailed conveyance system infrastructure data is provided with the model files to accompany this report.

Table 3.2 Summary of structure information within the conveyance system.

Data description	Number of Components in System	Length (ft)
Catch Basin	151	N/A
Manhole	33	
Pipe	218	24,942
Channel	N/A	9,661
Channel Cross-sections	93	N/A

3.1.2.5 Stormwater Management Facilities

A total of 9 stormwater management facilities (detention ponds) are included within the subject watershed. Critical information for modeling these facilities consists of elevation-area information and control structure information for creating discharge rating curves. Recall that sub-watersheds were delineated for each of these facilities. Stage-storage information was collected in the form of elevation-area information from contours on record drawings and from Town topographical information where record drawings were not available. Control structure information was collected both from record drawings and field data collection. Elevation information was adjusted using Town topographical information for consistency throughout the system. Stage-storage information for each facility serving each applicable area is provided in Appendix I.

3.1.3 Model Calibration

As part of the TOB modeling efforts, rain gage and flow monitoring equipment were installed throughout Town in January 2009. At the outfall of the subject watershed, a Teledyne ISCO 2150 Area-Velocity Module is used to measure velocity and depth at 15-minute increments. With the cross-sectional area of the channel known, the flow is automatically calculated using the Manning's equation. At the time of this research (August 2009), approximately 8 months of flow measurement data exists for the subject watershed, beginning in January, 2009. Several tipping bucket rain gauge are located within the TOB. The closest to the subject watershed is located approximately 1,500 feet from the watershed outfall and was used for rainfall data for the purpose of calibration. No significant rainfall events occurred (> 0.75 inches) until May of 2009. Between May and August, there were 6 occurrences where notable rainfall fell within a 24-hour period (see Table 3.3).

A review of Tables 3.1 and 3.3 demonstrates that the short time period of measured data did not provide a wide range of storm events in terms of recurrence interval as described in Table 3.1. However, this best available data was used to evaluate the model's sufficiency for the purpose of this research. A review of the measured data for each storm described in Table 3.3 reveals that two are not usable for calibration due to compromised data for the June 17th and the July 27th storms. With these storms, measured data was mixed with negative flow values. From discussion with TOB staff, it is suspected the compromised data is a result of debris caught on the monitoring equipment. The TOB plans to provide a cage device to attempt to reduce the potential of debris affecting future results.

Table 3.3 Notable measured rainfall in the subject watershed since flow monitoring began.

Date	Rainfall in 24-hour period (in)	Description of rainfall characteristics with 24-hour period	Comparable recurrence interval* (year, duration)
5/15/2009	0.91	0.87 inch as single centric peak intensity event, remainder sporadically throughout the 24-hour period	1-2-year, 20-30 min
5/26/2009	1.09	0.91 inch as a single event, remainder sporadically throughout the 24-hour period	< 1
6/17/2009	1.28	fell as two events about 12 hours apart, ground saturated from previous days, both low intensity except one burst w/ second event	< 1
7/17/2009	1.06	separated as two events about 7 hours apart, both centric peak intensity and brief	1-year, 10-15 min
7/27/2009	1.2	fell as a single event, nearly centric peak intensity	<1
8/5/2009	1.04	fell as a single event with early peak intensity	1-year, 1-hr and/or 2-year, 20-30 min

*** A recurrence interval of smaller duration may have been present within the 24-hour rainfall period.**

Using the actual rainfall hyetographs, as opposed to synthetic storm events, the remaining four storms from Table 3.3 were run through the subject watershed model. Volume and peak runoff at the outfall of the watershed is compared (see Table 3.4). Hydrographs are provided in Appendix F. Visual inspection of hydrograph comparisons provides satisfactory results for each storm except the 7/17/2009 storm. There are two observations that may explain the discrepancy between modeled and measured data for the 7/17/2009 storm:

- It is noted from the rainfall data on the hydrograph that one reading is much higher than the remainder of the storm at the 9th hour (about 0.5 inches measured for a 15 minute time increment). It is suggested that the intensity indicates thunderstorm cells which may have been highly variable in terms of the spatial variability of the rainfall, as opposed to longer duration storms that typically cover a larger geographical area. It is conceivable, that more intense rainfall was falling in the upper reaches of the watershed. The closest rain gauge used for this calibration is located just downstream of the watershed outfall.

- The 24-hour rainfall is noted to have fallen as two separate storms within the 24-hour period, about 7 hours apart. The SCS hydrology used in the model is calculating rainfall "stored" in the system based on a CN value with the first storm that occurs. This available storage is no longer available when the second storm occurs 7-hours later. In reality, storage in the form of infiltration, initial abstraction from canopy and depression storage may have again been available to a certain extent with the second rainfall. This could possibly explain the much higher peak seen in the second storm with the modeled results. In general, the SCS model hydrology is not suited for continuous simulation, or in this case, multiple defined storm events.

Table 3.4 Peak runoff rate and volume comparisons between measured and modeled flow data.

Storm event (data)	Peak (cfs)			Volume (ft^3)		
	Measured	Modeled	% Difference	Measured	Modeled	% Difference
5/15/2009	38.3	32.5	16.4	159,840	118,377	29.8
5/26/2009	25.0	24.4	2.4	191,386	184,561	3.6
7/17/2009	26.3	34.9	-28.1	165,639	190,153	-13.8
8/5/2009	26.8	36.4	-30.4	102,000	173,306	-51.8

Removing the 7/17/2009 storm for the reasons described above, the average percent difference between modeled and measured peaks and volumes for the remaining three storms is 16.8% and 28.4%, respectively. A comparison of time to peak finds the modeled and measured data occur at the same, or within one time increment (15 minute increments), in all cases. Considering the variability that occurs within an urban watershed and with the spatial distribution of rainfall itself, the results of the model are considered satisfactory in comparison with the measured data. No calibration, in terms of adjustment of variables (CN or t_C), is deemed justifiable, and the subject model is considered acceptable for the purpose of this research.

3.2 Modeling Scenarios & Application of Alternative Strategies

3.2.1 Applicable Areas

For the purpose of this research, applicable areas are defined as those drainage areas subject to the alternative management strategies that will be tested with the various scenarios described in Section 3.3. In general, these are drainage areas served by existing stormwater management facilities. However, since the watershed was mostly developed prior to the requirement for stormwater management, there are limited existing facilities, and good distribution throughout the watershed is not present. Therefore, four "dummy" facilities have been added in the model to provide adequate coverage and distribution for the purpose of this study (see Appendix G for mapping of applicable areas). These dummy facilities have been provided in locations where there is potential for retrofitting or redevelopment. All dummy ponds were designed to detain the post-development peak runoff rate to the predevelopment rate for the 2- and 10-year design storms, similar to what would have been the design requirements for the existing facilities within the pilot watershed.

3.2.2 Existing Regional Facilities

There are two facilities constructed as regional stormwater ponds within the study watershed. These ponds were constructed as capital improvement projects and were not associated with individual development. These ponds handle flows from large drainage areas that include a significant portion of the study watershed (e.g. Owens Park and Wong Park ponds accept flows from approximately 46% of the watershed). It would not be feasible to construct a pond at these locations to meet site specific stormwater management criteria intended for development projects, and these ponds were not originally designed in that manner.

Since these regional facilities potentially have a significant effect on the flows at the watershed outfall, each of these facilities will also be considered as points of interest when analyzing modeling results. With each management strategy routing scenario, the maximum water surface elevation (WSE) in these ponds will be noted and compared. The drainage areas to these ponds are not considered as applicable areas and strategies are not applied to these facilities in any of the modeled scenarios. However, in some cases, smaller drainage areas upstream are provided dummy facilities as described above.

3.2.3 Strategy Parameter Development

For *all* of the management strategies there is a component that can be considered a "baseline" target to be met. The designation of baseline targets for each strategy is discussed in this section. Several of the alternatives strategies to be represented in modeling scenarios require watershed-wide analysis to determine parameters critical to the implementation of the strategy, as discussed in Section 2.3. The watershed analysis for determination of each these parameters is discussed in this section.

3.2.3.1 Baseline Development

The baseline target is a reference either related to the predevelopment condition of the subject development site or a baseline condition at a point of interest (POI) within the overall watershed. For the purpose of this research, the baseline for configuration of *site based* strategies will assume the applicable area as meadow, with a HSG equal to C for computing runoff, resulting in a CN equal to 71. Similarly, the baseline for *watershed POI based* strategies will consider *all* applicable areas within the watershed as meadow (HSG = C) for computing runoff. Using a land cover condition of meadow in areas where stormwater management currently exists will approximate the period prior to stormwter management regulation within the watershed. In addition, the following modifications were made to the watershed model to simulate a watershed baseline condition:

- Stormwater facilities are set inactive in the model.

- Basins draining to management facilities were set to drain to the next downstream node unless it was appropriate to merge the basin with the next downstream sub-basin (e.g. adjacent applicable areas).
- Time of concentration was modified to represent a predevelopment condition (no storm sewer, etc.).

Using meadow as a consistent predevelopment condition subject for applicable areas allows for evaluation of all management strategies. The model was run with the baseline modifications described to determine baseline targets at the POI of the watershed, as shown in Table 3.5.

Table 3.5 Baseline (predevelopment) information at the POI for SCS 24-hour design storms.

Recurrence Interval (yr)	1-year	2-year	10-year	25-year
Peak flow (cfs)	157.72	252.05	500.69	721.00
Volume (ac-ft)	21.6	31.0	61.1	81.2
Time to peak (min)	738	732	738	732

In addition, for comparison purposes, an existing condition scenario and an existing condition scenario *without* development facilities was routed. The results are discussed in detail in Chapter 4. Table 3.6 and 3.7 provides the routing information for these scenarios that are described as:

- Scenario 1: POI Baseline - The watershed with all applicable areas are modified to the pre-developed condition as described above in this section. Some applicable areas were merged with the immediate downstream sub-basin and the overall CN adjusted accordingly. Time of concentration was modified to assume no channel flows. Results from this scenario will be used as watershed baselines for implementation of the watershed based management strategies.
- Scenario 2: Existing - The current condition of the subject watershed with the addition of the "dummy" facilities described in Section 3.2.1. This scenario demonstrates results based on the actual implementation of a traditional strategy (post to pre-peak runoff rate for the 2- and 10-year design storms) within the watershed. Although the design for the dummy facilities assumed a CN=71, the

predevelopment condition in for the existing facilities would vary based on the condition at the time of development.

- Scenario 3: No facilities - The current condition of the watershed with the stormwater management facilities serving applicable areas set as inactive in the model. Note that this only excludes facilities related to development and that the regional facilities remain active in the model. This scenario is provided for comparison purposes as a no management option. No changes are made to the land cover conditions so that the existing condition with no management is reflected in results.

Hydrographs for the three scenarios described above can be found in Appendix H, and include a plot illustrating:

- The predevelopment (POI baseline) hydrographs for the 1-, 2-, 10- and 25year design storms (Scenario 1).
- The existing condition hydrographs for the 1-, 2-, 10- and 25year design storms (Scenario 2).
- The existing condition with no detention hydrographs for the 1-, 2-, 10- and 25year design storms (Scenario 3).
- Comparison of Scenarios 1, 2 and 3 for the 1-, 2-, 10- and 25year design storms.

Table 3.6 Peak runoff rates and runoff volumes at the outfall of the watershed for Scenarios 1 through 3.

Scenario 1: POI Predevelopment				
Recurrence Interval	1-year	2-year	10-year	25-year
Peak flow (cfs)	157.72	252.05	500.69	721.00
Volume (ac-ft)	21.6	31.0	61.1	81.2
Scenario 2: Current (existing) condition				
Recurrence Interval	1-year	2-year	10-year	25-year
Peak flow (cfs)	186.76	285.43	527.46	705.55
Volume (ac-ft)	26.6	37.9	71.3	92.4
Scenario 3: Current (existing) condition with development detention removed				
Recurrence Interval	1-year	2-year	10-year	25-year
Peak flow (cfs)	197.54	350.07	567.37	743.57
Volume (ac-ft)	25.1	40.5	67.1	87.0

Table 3.7 Baseline (predevelopment) information at the regional facility points of interest.

		Owens St. Pond	Lower Wong Pond
POI-Pre	1-year	2092.44	2077.05
	2-year	2092.95	2077.89
	10-year	2094.65	2078.39
	25-year	2095.44	2078.50
Existing	1-year	2092.54	2077.47
	2-year	2093.01	2078.04
	10-year	2094.64	2078.40
	25-year	2095.38	2078.50
No facilities	1-year	2092.70	2077.50
	2-year	2093.30	2078.04
	10-year	2095.04	2078.49
	25-year	2095.73	2078.50

3.2.3.2 Unit Release Rate

The previous section describes the watershed analysis that provides the baseline runoff release rate for the watershed used to determine unit release rates for development under the URR strategy. Recall from Section 2.3.1.1, the MMSD has set the following release rates for stormwater management based on a watershed study to maintain existing flow rates for a specified baseline:

- ✓ A release rate of 0.5 cfs/acre for the one percent probability event (100-year recurrence interval)
- ✓ A release rate of 0.15 cfs/acre for the fifty percent probability event (2-year recurrence interval)

The MMSD rates were set based on maintaining a specified runoff rate at a point of interest through an iterative watershed modeling analysis. In the case of MMSD, the baseline was established based on peak flows for 1980 conditions at the point of interest. Similarly, release rates for the purpose of this study are also set to maintain the POI predevelopment runoff rates provided in Table 3.5 for the 2- and 25-year recurrence intervals. MMSD's designation of a release rate for the 2- and 100-year storm is intended to provide protection for a wide range. Since the 100-year design storm is not analyzed in this study, release rates for the subject watershed are set for the 2- and 25-year recurrence intervals.

With the baseline peak release rate at the POI known (Table 3.5), an iterative process was used to determine the unit release rates for the subject watershed that would result in a peak runoff rate at the POI closest to the baseline POI-predevelopment peak runoff rates. This process entailed the redesign of each pond serving an applicable area for a variety of release rates for the 2- and 25-year recurrence interval. The release rate for each recurrence interval that produced the peak at the POI that fell closest to the baseline target was selected. After multiple iterations using varied unit release rates, the following met the objective for the subject watershed:

- ✓ A release rate of 2.00 cfs/acre for the twenty-five percent probability event (25-year recurrence interval)
- ✓ A release rate of 0.25 cfs/acre for the fifty percent probability event (2-year recurrence interval)

Based on the determined unit release rates for the subject watershed, each applicable area is assigned an allowable release rate based on area. With this, the control structure for each pond serving an applicable area was modified using an orifice for the 2-year storm and a riser for the 25-year storm to achieve peak runoff rates from each applicable area at, or just less than, the allowable discharge (see Table 3.8a and 3.8b). Resulting peak runoff rates for the 2- and 25-year design storms for the overall watershed are 268.5 cfs and 675.5 cfs, respectively (see Table 3.9). Note that each is less than the peak runoff rates for the overall watershed in the pre-developed condition.

Table 3.8a Information for application of the determined unit release rates for the 2-year design storm. *Area* reflects area of applicable area; *allowable release rate* is the area multiplied by the URR and *design* value is the actual release rate after modification of the control structure.

Site ID	Area (ac)	2-yr URR (cfs/acre)	2-yr Allowable Release rate (cfs)	Design (cfs)
A	30.27	0.25	7.57	7.54
B	5.97	0.25	1.49	1.31
C	7.56	0.25	1.89	1.89
D	6.05	0.25	1.51	1.51
E	21.53	0.25	5.38	4.80
F	3.98	0.25	1.00	0.83
G	2.35	0.25	0.59	0.59
H	2.07	0.25	0.52	0.49
I	8.77	0.25	2.19	1.92
Totals	88.55	N/A	22.14	20.88

Table 3.8b Information for application of the determined unit release rates for the 25-year design storm. *Area* reflects area of applicable area; *allowable release rate* is the area multiplied by the URR and *design* value is the actual release rate after modification of the control structure.

Site ID	Area (ac)	25-yr URR (cfs/acre)	25-yr Allowable Release rate (cfs)	Design (cfs)
A	30.27	2.00	60.538	58.89
B	5.97	2.00	11.94	10.85
C	7.56	2.00	15.12	14.15
D	6.05	2.00	12.104	10.51
E	21.53	2.00	43.052	41.23
F	3.98	2.00	7.96	7.05
G	2.35	2.00	4.7	4.7
H	2.07	2.00	4.132	3.76
I	8.77	2.00	17.546	16.49
Totals	88.55	N/A	177.09	167.63

Table 3.9 Comparison of peak outflow at the watershed POI between the baseline predevelopment peak runoff rates and with the unit release rates applied to the watershed.

2-year peak runoff rate			25-year peak runoff rate		
Tailored URR (cfs)	POI-Predevelopment (cfs)	% difference	Tailored URR (cfs)	POI-Predevelopment (cfs)	% difference
267.22	252.05	6.01	689.61	721	4.36

Details regarding model representation of this strategy to applicable areas are discussed further in Section 3.3.2.1.

3.2.3.3 Stratified Release Rate Approach

Recall from Section 2.3.2 that a stratified approach to stormwater management is intended to meet the goal of no increase in maximum peak flows at a point, or points, of interest by implementing management requirements at the site scale based on a watershed analysis. The watershed analysis performed in Section 3.2.3.2 that found baseline runoff release rate for the watershed is used for implementation of this strategy, specifically the calculation of release rates. For the watershed, release rates were developed for each applicable area using the following equation:

$$R_{ij} = \left(\frac{Q_{ij}}{Q_i} \right) \times 100 \qquad (3\text{-}3)$$

Where: R_{ij} = release rate of sub-basin i at downstream reach j (% of predevelopment peak runoff rate)

Q_{ij} = predevelopment discharge contribution of sub-basin i to the peak discharge of a downstream reach j (cfs)

Q_i = predevelopment peak discharge of sub-basin i (cfs)

The variables in Equation 3-3 were determined for the watershed by comparing the runoff hydrograph at the POI to the predevelopment hydrograph for each applicable area. This task was accomplished with the following steps:

1. Model modification: The predevelopment condition for each applicable area was assumed to have a CN equal to 71. Time of concentration was also modified to take into account the lack of channel flow.
2. For the applicable areas in Step 1, a separate model for each applicable area (refer to Appendix G) was generated that included only the respective applicable area and all of the conveyance system that conveyed flows from that specific applicable area to the watershed outfall.
3. Each model from step two (applicable areas A through I) was run and the time versus outflow data (3 minutes increments) at the watershed outfall was entered into a spreadsheet.
4. The predevelopment POI model time versus outflow data (3 minutes increments) is entered into the spreadsheet from step 3.
5. Q_{ij} was then determined as the flow for each specific applicable area model (from step 2) that corresponds with the time step generating the *peak* flow for the overall watershed. Q_i is simply the peak flow rate for the individual applicable area.

Table 3.10 Values used for generation of release rates for applicable areas: 2-year storm.

	A	B	C	D	E	F	G	H	I
Q_i	15.9	4.3	6.2	4.3	19.5	3.0	1.6	1.6	5.9
Q_{ij}	0.2	0.2	4.9	0.2	13.6	2.6	1.5	1.3	0.4
R_{ij}	1.1	5.1	80.0	4.4	69.6	84.8	96.9	84.1	6.3
Allowable release - % of pre-peak	100.0	100.0	80.0	100.0	70.0	80.0	100.0	80.0	100.0

Table 3.11 Information used for generation of release rates for applicable areas: 10-year storm.

	A	B	C	D	E	F	G	H	I
Q_i	42.1	11.0	15.7	11.1	42.8	7.6	4.2	4.0	15.4
Q_{ij}	6.5	3.2	12.5	1.6	35.9	5.9	3.4	2.2	2.1
R_{ij}	15.4	28.8	80.1	14.8	83.8	76.9	81.2	55.9	13.6
Allowable release - % of pre-peak	100.0	100.0	80.0	100.0	80.0	80.0	80.0	60.0	100.0

Table 3.12 Information used for generation of release rates for applicable areas: 25-year storm.

	A	B	C	D	E	F	G	H	I
Q_i	61.9	16.0	22.7	16.1	59.5	11.0	6.1	5.7	22.6
Q_{ij}	7.0	4.4	18.5	2.4	37.1	9.9	5.5	4.0	2.6
R_{ij}	11.3	27.5	81.2	14.6	62.2	89.4	90.5	69.4	11.5
Allowable release - % of pre-peak	100.0	100.0	80.0	100.0	60.0	90.0	90.0	70.0	100.0

The steps above resulted in the release rate values for the 2-, 10- and 25-year design storms as shown in Table 3.10, 3.11 and 3.12, respectively. With release rates determined, rules from Section 2.3.2.1 were used to assign the release rate for each applicable area. When comparing the generated allowable release rates for each applicable area (reference Appendix G), the following observations are noted:

- Applicable areas with discharge of flows that are not subjected to regional management facilities downstream, all appear to contribute more significantly to the watershed peak runoff rate than those that flow through regional facilities downstream. In fact, all applicable areas that discharge flows that are subjected to downstream regional facilities are assigned an allowable release rate of 100% of the predevelopment peak runoff rate. This indicates that the regional facilities delay flows from these applicable areas enough to mitigate some of their impact to the watershed peak runoff rate.
- Applicable areas that do not have discharge subjected to regional stormwater management downstream appear to contribute more than those that do no matter where they are located within the watershed.

- Applicable area E, with the lowest assigned release rates on average for the range of design storms, appears to indicate that the size of the sub-basin (not only the locations within the watershed) may influence designation of release rates.
- Assigned release rate can vary for sub-basins within close proximity of each other. This indicates that timing is sensitive within the relatively small and urbanized watershed.

Note that a collection of unit release rates would typically be generated for all sub-basins. These rates would be used to generate a map by merging neighboring values for a generalized map that groups sub-basins within a watershed and assigns them specified release rates. However, since only applicable areas are subject to stormwater management for the purpose of this research, a generalized map is not created, and the release rates generated using Equation 3-3 are applied for modeling this strategy.

Details regarding model representation of this strategy to applicable areas are discussed further in Section 3.3.3.1.

3.2.3.4 Volumetric Design Procedure

In order to represent the VDP, a watershed-wide analysis to determine the critical storm period is required (refer to Section 2.3.3.3). The critical time period is defined as the amount of time from the peak of the design rainfall to the peak of the runoff hydrograph resulting from a subject watershed at some critical point (the outfall of the overall watershed for the purpose of this research). For multiple critical locations within a watershed, a critical time period would be determined for each. Site stormwater management would be based on the critical time period assigned to the closest downstream critical location. The MMSD VDP requires that post-development runoff volumes shall be limited to the predevelopment runoff volumes during a critical time period for the 2- and 100-year storm. Since the 100-year design storm is not analyzed in this study, release rates for the subject watershed are set for the 2- and 25-year recurrence intervals, similar to the URR analysis.

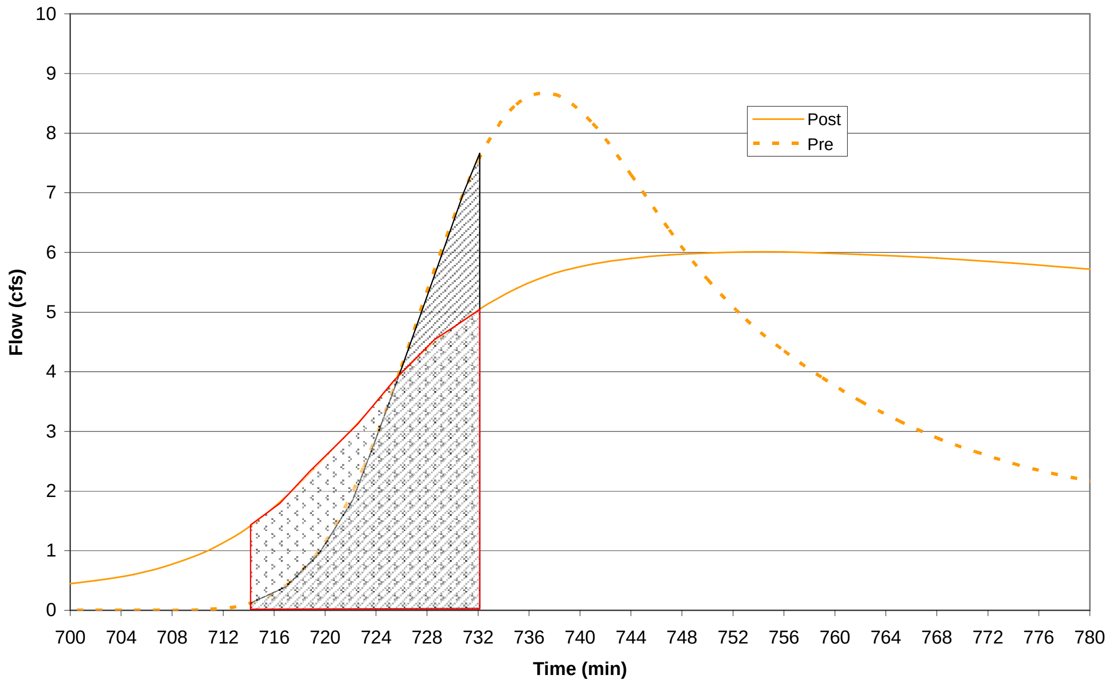


Figure 3.2 Graphical depiction that illustrates the VDP requirement to maintain the predevelopment volume during the critical time period. The shaded area beneath the "post" hydrograph must be less than the shaded area beneath the predevelopment hydrograph. The critical time period as shown is the time from 714 - 732 minutes.

Determination of the critical time period simply involved identifying the time of peak intensity from the design rainfall. This peak intensity is determined to be at 714 minutes for both the 2- and 25-year SCS, Type II, 24-hour design storms. Next, the predevelopment POI model scenario was used to determine the time of peak runoff at the watershed outfall. Peak runoff rate at the watershed outfall occurs at 732 minutes. Therefore, design of facilities serving applicable areas shall maintain post-development runoff volume to the predevelopment volume for the 18 minutes during the critical time period between 714 and 732 minutes.

Details regarding model representation of this strategy to applicable areas are discussed further in Section 3.3.4.3.

3.2.4 Stormwater Management Modeling Scenarios

The traditional and alternative stormwater strategies are represented as unique model scenarios assuming all applicable areas are subject to the specific strategy. For comparison, a "no-management" model scenario is run with the existing conditions represented to demonstrate the case of no stormwater management within the watershed. It is noted that a scenario is defined for the purpose of this report as a unique model run for a specified strategy represented in the watershed. Strategies to be modeled, described in detail in Sections 2.2 and 2.3, include the:

1. traditional peak reduction;
2. tradition peak reduction with treatment component;
3. unit release rates;
4. stratified release rate;
5. Low Impact Development;
6. Full Spectrum Detention;
7. Volumetric Design Procedure

Each scenario is subjected to a range of design storms. Control structures for ponds serving each applicable area are designed to meet the design specifications for each of the strategies. Therefore, outflow hydrographs from each applicable area are representative of the outflow that would be expected to result from the specific strategy. Methodology for achieving representative outflow for each strategy is described in Section 3.3.

3.3 Alternative Strategy Model Representation

Provided in Section 3.3 are the methodologies for representing each management strategy within the model. Each sub-section provides a description of the steps required to generate a hydrograph from each applicable area representative of the subject strategy. Each section references Appendices that provide detailed information related to the design of facilities required for each strategy. Common in all cases are the design storms, the pre- and post development hydrological parameters and the facility stage-storage information that serve each applicable area. A summary of each strategy is provided in Table 3.13. The following common information can be found in Appendix I:

- SCS, Type II, 24-hour design rainfall for the 1-, 2-, 10-, and 25-year design storms.

- Pre- and post-development hydrologic parameters for each applicable area.
- Stage-storage information for the facility serving each applicable area.

Table 3.13 Summary of strategies and their general characteristics.

Management Strategy	Design Storm focus at Site Scale	Design Storm focus at Watershed Scale	Other Unique Component
Traditional	2, 10	N/A	
Traditional with Water Quality Component	2, 10	N/A	30-hour drawdown of WQ volume
Unit Release Rates	N/A	2, 25	release rate per acre to maintain condition at POI
Stratified Release Rate	N/A	2, 10, 25	release rate ratio based on contribution at POI
Low Impact Development	2, 10	N/A	Maintain runoff from LID storm (1-year)
Full Spectrum Detention	10, 25	N/A	72-hour drawdown of the EURV + 10%
Volumetric Design Procedure	N/A	2, 25	volume detention during critical time period

3.3.1 Traditional Stormwater Management

Recall from Section 2.2 that traditional management strategies typically require post development peak runoff rates not to exceed the predevelopment peak runoff rate for specified design storms. In more recent years, a treatment component was also added that can potentially affect the runoff hydrograph. Model representation of traditional strategies is discussed in this section.

3.3.1.1 Maintaining Predevelopment Peak Runoff Rate

As previously mentioned in Section 3.2.3.1, for the purpose of this research, the baseline (predevelopment) for configuration of *site based* strategies will consider the applicable area as meadow (HSG = C) for computing runoff, resulting in a CN equal to 71. Also, time of concentration is modified for each applicable area to simulate a predevelopment condition by not considering any paved channel flow to be present.

With baseline predevelopment parameters established, a predevelopment peak runoff rate is generated for each design storm and for each applicable area. These rates are set as the target runoff rate and the outflow control structure serving each applicable area is modified allow discharge to match, or fall just below, the predevelopment runoff rate. For the 2- and 10-year design storms, a combination of orifice(s) and a riser is used to detain to the predevelopment runoff rate. A weir will be provided with its crest set just above the maximum water surface elevation of the 10-year storm to allow passage of storms that exceed the design storms, when needed. This design is similar to the approach taken by a design engineer and results in an accurate depiction of the strategies outflow hydrograph from each applicable area. Typical of traditional requirements for this strategy, design of control structures will reduce the 2- and 10-year design storms.

The following information associated with the representation of this strategy can be found in Appendix J:

- Control Structure information for each facility serving each applicable area.
- Pre- and post development hydrographs for each applicable area for design storms that meets the strategy requirements.

3.3.1.2 Treatment Component Added to Peak Rate Reduction

Section 2.2 discusses the addition of a treatment component to the traditional strategy in the previous section. This component currently requires the treatment of the "first flush" in Virginia. The first flush is defined as the first 1/2-inch of runoff from the impervious cover within an applicable area, also known as the treatment volume.

The treatment volume can be treated using a variety of BMP practices. Depending on BMP selection, the resulting post-development hydrograph from a development could vary. For example, an infiltration BMP would subtract some volume of runoff from the outflow hydrograph. A filtering device may not affect the hydrograph at all. Other practices such as bioretention would have some minimal affect on the outflow hydrograph depending on the infiltration rate on the soil mixture and storage capacity in the basin. In Blacksburg, the location of the subject watershed, infiltration is typically not feasible due to clayey soils.

To represent this strategy for the purpose of this research, extended detention (pollutant settling) is used for the treatment of the first flush. This BMP option is a typical selection in Blacksburg and can also be a feasible option for the retrofit of existing facilities to provide a quality benefit. In Virginia, extended detention requires a minimum water quality volume drawdown of 30-hours (e.g. the outlet is sized to drain the water quality volume in 30-hours). Therefore this strategy will be represented for each applicable area by providing this drawdown for the water quality volume. Impervious area within each applicable area is generated from the GIS land cover data discussed in Section 3.1.2.1. The water quality volume is calculated as 1/2-inch times the impervious area in the applicable area.

A primary, or water quality orifice, is used for the water quality drawdown. With this being an additional component to the traditional strategy discussed in the previous section, representation of this model will also include post-peak runoff rate reduced to the predevelopment peak runoff rate for the 2- and 10-year storms. A secondary orifice will be used for the 2-year peak runoff design and a riser or orifice for the 10-year, if needed. In addition, a weir will be provided with its crest set just above the maximum water surface elevation with routing of the 10-year storm to allow passage of storms that exceed the design storms.

The following information associated with the representation of this strategy can be found in Appendix K:

- Calculation of water quality volume including the corresponding stage within the stormwater facility.
- Water quality volume drawdown hydrograph
- Control Structure information for each facility serving each applicable area.
- Summary Table providing comparison of pre- and post-development peak runoff rates with the Traditional with Water Quality Component strategy implemented at each applicable area.
- Pre- and post development hydrographs for each applicable area for design storms that meets the strategy requirements.

3.3.2 Downstream peak Focus Blanket Strategy

Of the alternative strategies evaluated for this research, one can be considered a strategy that applies a blanket strategy for site stormwater release requirements based on a watershed perspective. This perspective is the goal of maintaining a maximum release rate established at a downstream point of interest. This strategy is the Unit Release Rate Strategy and is described in more detail in Section 2.3.1.1. With this research, the point of interest is considered the outfall of the overall watershed. The maximum allowable downstream release rate at the outfall was established with the representation of a predevelopment baseline peak runoff rate as described in Section 3.2.3.1.

3.3.2.1 Unit Release Rate

Model representation of the URR strategy inherently requires a watershed analysis to determine unit release rates that maintain a condition at the point of interest (outfall of watershed in the case of this research). As described in more detail in Section 3.2.3.2, the analysis evaluated a range of unit release rates on all applicable areas for the 2- and 25-year storm events to designate the unit release rates that achieved the downstream objective. The unit release rates for the subject watershed were found to be:

- ✓ A release rate of 2.00 cfs/acre for the four percent probability event (25-year recurrence interval)
- ✓ A release rate of 0.25 cfs/acre for the fifty percent probability event (2-year recurrence interval)

Model representation is based on maintaining the above unit release rates. Maximum allowable release from each applicable area is determined by multiplying the release rate by the area of the applicable area. The control structure serving each applicable area is then modified to achieve the allowable maximum release rate. Similarly as described for the traditional strategy, an orifice is used to maintain the release rate for the 2-year storm and a riser structure for the 25-year storm. The actual control structure design is performed as part of the analysis described in Section 3.2.3.2. Results are found in Table 3.8 that includes the applicable area areas, the allowable release rates and the actual design release rates after control structures have been modified.

The following information associated with the representation of this strategy can be found in Appendix L:

- Table providing the maximum allowable release rates per applicable area for both the 2- and 25-year design storms.
- Control Structure information for each facility serving each applicable area.
- Pre- and post development hydrographs for each applicable area for design storms that meets the strategy requirements.

3.3.3 Downstream peak Focus Stratified Strategy

Of the alternative strategies evaluated for this research, one can be considered a strategy that applies a stratified approach for site stormwater release requirements based on a watershed perspective. This perspective is the goal of maintaining a maximum release rate established at a downstream point of interest. This strategy is the Stratified Release Rate strategy and is described in more detail in Sections 2.3.2.1 and 2.3.2.2. For the purpose of this research, the point of interest is considered the outfall of the overall watershed. The maximum allowable downstream release rate at the outfall was established with the representation of a predevelopment baseline peak runoff rate as described in Section 3.2.3.1.

3.3.3.1 Stratified Release Rates

Model representation of the Stratified Release Rate strategy inherently requires a watershed analysis to determine release rates based on the contribution of sub-areas within the watershed to the peak runoff rate of the overall watershed. As described in more detail in Section 3.2.3.3, the analysis evaluates each sub-basin's contribution to the watershed's overall peak and determines what percentage of the predevelopment peak runoff rate within the sub-basin is the allowable release rate for the developed condition. Results of the analysis can be found in Tables 3.10, 3.11 and 3.12. Based on the percentages provided in these tables, Table 3.13 provides the allowable release rate for each applicable area for the 2-, 10- and 25-year storm events, respectively. It should be noted that the LVPC implementation of a stratified strategy requires post-development peak reduction to 30% of the predevelopment peak for the 2-year design storm. However, for representation of this strategy for the purpose of this research, the %-allowable methodology is used for all design storms.

Model representation is based on maintaining the release rates provided in Table 3.14. The control structure serving each applicable area is modified to achieve the allowable maximum release rate. A primary orifice is used to maintain the release rate for the 2-year storm, a secondary orifice for the 10-year storm and a riser structure for the 25-year storm, if needed.

The following Information associated with the representation of this strategy can be found in Appendix M:

- Table providing the maximum allowable release rates per applicable area.
- Control Structure information for each facility serving each applicable area.
- Pre- and post development hydrographs for each applicable area for design storms that meets the strategy requirements.

Table 3.14 Allowable release rates per applicable area for the 2-, 10- and 25-year storms. Allowable release % and predevelopment runoff values determined as described in Section 3.2.3.3.

Applicable Area	Predevelopment Peak Runoff Rate (cfs)			Allowable release (% of predevelopment)			Allowable release rate (cfs)		
	2yr	10yr	25yr	2yr	10yr	25yr	2yr	10yr	25yr
A	15.9	42.1	61.9	100	100	100	15.9	42.1	61.9
B	4.3	11.0	16.0	100	100	100	4.3	11.0	16.0
C	6.2	15.7	22.7	80	80	80	5.0	12.6	18.2
D	4.3	11.1	16.1	100	100	100	4.3	11.1	16.1
E	12.3	32.3	47.5	80	100	80	9.8	32.3	38.0
F	3.0	7.6	11.0	80	80	90	2.4	6.1	9.9
G	1.6	4.2	6.1	100	80	90	1.6	3.4	5.5
H	1.6	4.0	5.7	80	60	70	1.3	2.4	4.0
I	5.9	15.4	22.6	100	100	100	5.9	15.4	22.6

3.3.4 Volume Focus Strategy

Of the alternative strategies evaluated for this research, three can be considered strategies that apply a volume focused approach for site stormwater release requirements. Two of these strategies, Low Impact Development and Full Spectrum Detention, are considered more site than watershed based strategies since they do not require a watershed analysis for implementation (reference Sections 2.3.3.1 and 2.3.3.2, respectively). However, the third of the volume based strategies, the Volumetric Design Procedure (Section 2.3.3.3), does require a watershed analysis to determine the critical storm period. Although only Low Impact Development actually requires a reduction in volume from leaving a development site, the Full Spectrum Detention and Volumetric Design Procedure each focus on the extended detention of a specified volume, taking the focus off of peak reduction as with other strategies evaluated as part of this research.

3.3.4.1 Low Impact Development

LID is intended to guide design that maintains or restores a development to the predevelopment hydrologic and hydraulic conditions. The optimal result is that the post-development runoff hydrograph matches the predevelopment hydrograph by using a combination of retention (infiltration) and detention along with design components that maintain the time of concentration.

As mentioned in Section 2.3.3.1, it is not the intention of this research to attempt to provide micro-modeling guidance at the site scale, which would be the case when attempting to represent an integrated LID site design using traditional routing and computational methods. In order to represent an LID site, a designer would typically:

1. Design the site to lower the CN by breaking out the site into discrete areas to account for design features (such as maintaining wooded areas), therefore generating an LID CN.
2. Provide design features to maintain the post-development time of concentration to the predevelopment time of concentration.
3. Use the LID chart series described in Section 2.3.3.1 to determine the area of the site needed for retention to maintain predevelopment volume.
4. Use the LID chart series described in Section 2.3.3.1 to determine the volume needed for detention and/or retention to maintain the predevelopment peak.

The four steps described above would be followed to result in a site design that maintains the predevelopment runoff hydrograph for an LID design storm which is defined as the greater rainfall between either the:

- rainfall at which direct runoff begins from a site that assumes a CN based on woods in good condition, with the existing hydrologic soils group (HSG). Equation 2.2 & 2.4 from TR-55 (initial abstraction) is used to determine the rainfall needed to initiate runoff, with a modifying factor; or
- the 1-year, 24-hour storm event.

To represent this strategy for each applicable area, it is assumed that steps 1 through 4 above have been implemented in a sufficient manner in that the post-development runoff hydrograph has been maintained to the equivalent of the predevelopment hydrograph for the LID design storm. This is accomplished in the model representation by providing storage beneath the primary orifice and by maintaining the predevelopment time of concentration. In all cases for each applicable area, maintaining the predevelopment volume resulted in a post-development peak less than the predevelopment peak, satisfying step #4 as listed above. A secondary orifice, if needed, and a riser structure is used to assure post-development peak runoff rates are maintained for the 2- and 10-year design storms, respectively since these requirements would also need to be met under Virginia Stormwater Regulations if this strategy was implemented. It should be noted that maintaining the predevelopment hydrograph for the LID design storm would be challenging, if not impossible in Blacksburg since infiltration is not a feasible method for volume reduction. However, other BMPs, such as rainwater harvesting, may provide some potential for meeting this goal.

The LID design storm is determined for each applicable area as the greater of either storm described above. To determine which is greater, Equation 2.2 and 2.4 from TR-55 (see Equation 3-4 below) is used to determine the rainfall needed to initiate runoff for each applicable area. Since it has previously been established that the assumed predevelopment CN for all applicable areas is 71, using Equation 3-4 below, initial abstraction (I_a) is equal to 0.82 inches. Therefore, 0.82 inches represents the rainfall at which direct runoff begins from an applicable area. From Table 3.1, it is found that the rainfall depth for the 1-year, 24-hour event is 2.31 inches. Since this is the greater of either storm described above, the 1-year, 24-hour event is the LID design storm.

$$I_a = 0.2S, \text{ where } S = \frac{1000}{CN} - 10 \quad (3\text{-}4)$$

The following information associated with the representation of this strategy can be found in Appendix N:

- Information for revised stage-storage information that provides storage beneath the primary orifice for maintaining the predevelopment runoff hydrograph for the LID design storm.
- Control Structure information for each facility serving each applicable area.
- Pre- and post development hydrographs for each applicable area for design storms that meets the strategy requirements.

3.3.4.2 Full Spectrum Detention

The unique component for model representation of the Full Spectrum Detention strategy is the extended detention of the excess urban runoff volume (EURV) which is defined as the difference between the post- and pre-development runoff volume. As discussed in Section 2.3.3.2, when the EURV plus 10% is captured and released over 72-hours, the runoff volume exceeding the EURV approximates the predevelopment volume. To simulate FSD, applicable areas are subject to detention that met the following runoff requirements:

- Detention and release of the EURV over a 72-hour drawdown period.
- Control of the 25-year peak runoff rate to the predevelopment peak (Note that the UDFCD requires control of the 100-year peak runoff rate; however, for the purpose of this research the 10- and 25-year storm is detained instead. The 10-year to stay consistent with Virginia Stormwater Regulation and the 25-year to

maintain the large storm detention requirement associated with the UDFCD criteria).

To meet the above requirements, the EURV is determined for each applicable area. To assist designers, a spreadsheet is provided by the UDFCD (reference Section 2.3.3.2). However, this spreadsheet utilizes Horton's Equation to account for volume losses and since predevelopment condition for the purpose of this research has been established for each applicable area as described in Section 3.2.3.1 using the SCS CN methodology, the UDFCD spreadsheet is not used.

To determine the EURV, methodology similar to the UDFCD method is performed using the following steps:

1. For the 1-, 2-, 10- and 25-year design storms, the predevelopment runoff volume was subtracted from the post-development volume.
2. The difference in volume between pre- and post for each storm and each applicable area is then divided by the area of the corresponding applicable area. Units are converted to provide results in depth (inches).
3. The impervious area for each applicable area is obtained. Percent impervious of each applicable area is calculated.
4. A plot is then created displaying % impervious versus the differences in volumes from step 1 for each design storm.
5. A trend line based on the averages of all results is created that will serve as the benchmark for the selection of the EURV based on an applicable areas % imperviousness (see Figure 3.3).

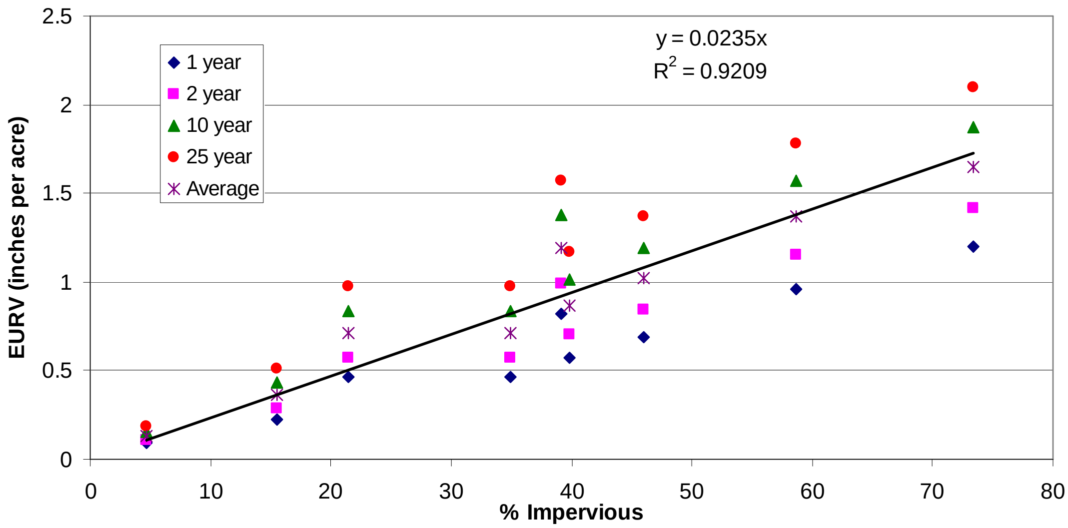


Figure 3.3 Establishment of the EURV was generated using data from all applicable areas for a range of design storms.

The EURV for each applicable area is generated using the trend line in Figure 3.3 and the areas of each applicable area (see Table 3.15). The EURV plus 10% is used to find a corresponding elevation from the stage-storage information for each facility serving each applicable area. Design of control structures assures that this volume has a 72-hour drawdown per the FSD design criteria. A primary orifice will detain the EURV volume while a secondary orifice and/or a riser will be used to detain to the predevelopment runoff rate for the 10- and 25-year design storms, respectively.

Table 3.15 EURV for each applicable area is calculated based on the EURV as inches per acre established in Figure 3.3.

Applicable Area	% Impervious (x)	EURV as inches per acre (0.0235x)*	Area (acres)	EURV (ft^3)
A	34.93	0.82	30.27	90,199
B	39.83	0.94	5.97	20,283
C	46.05	1.08	7.56	29,695
D	21.42	0.50	6.05	11,058
E	15.51	0.36	21.53	28,484
F	4.65	0.11	25.51	10,122
G	58.66	1.38	2.35	11,760
H	73.41	1.73	2.07	12,938
I	39.13	0.92	8.77	29,282

*** From Figure 3.1**

The following information associated with the representation of this strategy can be found in Appendix O:

- EURV plus 10% stage for each facility serving each applicable area.
- Control Structure information for each facility serving each applicable area.
- Hydrograph depicting drawdown of the EURV for each applicable area.
- Pre- and post development hydrographs for each applicable area for representative design storms.

3.3.4.3 Volumetric Design Procedure

Model representation of the VDP strategy inherently requires a watershed analysis to determine the critical storm period. Recall that the critical time period is defined as the amount of time from the peak of the design rainfall to the peak of the runoff hydrograph resulting from a subject watershed at some critical point (the outfall of the overall watershed for the purpose of this research). For model representation, MMSD requires the post-development runoff volumes shall be limited to the predevelopment runoff volumes during the critical time period for the 2- and 100-year storm. Since the 100-year design storm is not analyzed in this study, release rates for the subject watershed are set for the 2- and 25-year recurrence intervals. The critical time period was previously determined to extend from 714 minutes to 732 minutes.

For model representation of this strategy, design of facilities serving applicable areas shall maintain post-development runoff volume to the predevelopment volume for the 18 minutes during the critical time period between 714 and 732 minutes. This is accomplished using the following steps:

- Calculate the total volumes of runoff released during the critical time period from the predevelopment hydrographs for each applicable area for both the 2- and 25-year events. This sets the allowable volume discharge during the critical time period for each applicable area (see Table 3.16).
- Employ an iterative process by routing post-development hydrographs through modified control structures for each applicable area facility until the VDP objective is met.

Table 3.16 Allowable release volumes during the critical time period.

Applicable Area	2-year predevelopment volume released during the critical time period (ft^3)	25-year predevelopment volume released during the critical time period (ft^3)
A	11,504	53,497
B	3,793	15,464
C	5,508	21,255
D	3,768	15,460
E	9,600	43,048
F	2,675	10,707
G	1,400	5,827
H	1,388	5,555
I	5,202	21,667

In each case for each applicable area, a primary orifice detains the 2-year volume to the predevelopment volume during the critical time period volume while a riser is used as a secondary release for meeting the VDP requirement for the 25-year design storm.

The following information associated with the representation of this strategy can be found in Appendix P:

- Control Structure information for each facility serving each applicable area.
- Comparison of pre- and post development volumes released during the critical time period for the 2- and 25-year design storms.
- Pre- and post development hydrographs for each applicable areas for representative design storms.

4 Modeling Results

The overall objective of this research is to investigate downstream runoff peak reductions resulting from the application of alternative stormwater management strategies within the subject watershed. To achieve this goal, model routing results for design storms based on alternative management scenarios are compared. Chapter 4 presents the results of the modeled scenarios described in the previous chapters and is intended to address the following objectives as stated in Section 1.2:

- Evaluate characteristics of runoff hydrographs resulting from each applicable area with each alternative management strategy. Routing will be subject to a range of design storms.
- Analyze the resulting hydrographs at the watershed outfall generated from implementation of each alternative management strategy for all applicable areas within the watershed.

4.1 Runoff at the Site Scale

Results are reviewed in context of the components of the runoff hydrograph for each of the alternative management strategies over the range of design storms. This section discusses these metrics prior to their inclusion in the analysis of results for the overall watershed beginning in the subsequent section.

4.1.1 Strategy variations

All of the management strategies presented within this research have explicit variations to design parameters in regards to the following:

- Specified design storms required to be analyzed as part of analysis.
- Either a site based or watershed based focus for defining a target release rate for specified design storms.

Specified design storms require a predevelopment condition to be maintained explicitly either at the outfall of the site or at the outfall of the watershed, as determined by a watershed analysis. Table 4.1 summarizes those strategies based on site outfall peak reduction requirements and includes both Traditional strategies, LID and FSD. Also summarized in Table 4.1 are those strategies that are based on maintaining a predevelopment condition within the overall watershed, these include the Stratified Release Rate, URR and VDP strategies. However, where all of the site based strategies have at least part of their focus on detaining predevelopment peak runoff rates for specified design storms, only one of the watershed strategies (URR) also hinges on reduction of post-peak runoff rate to the predevelopment peak runoff rate at the watershed point of interest. Recall the difference from the site based strategies in this sense is that unit release rates per acre are developed with a watershed analysis to maintain the predevelopment conditions at the watershed outfall. The Stratified strategy is similar in that post-development peak runoff rates shall be detained to below a specified level for specific design storms, but varies in that the target runoff rate is a ratio of the predevelopment peak runoff rate based on a sub-basins contribution to the watershed peak runoff rate. Of the strategies presented, only the VDP management requirements have no requirement for detention of peak runoff rates. However, management is related to specified design storms, whereas volumes during the critical time period shall be maintained for these storms.

Table 4.1 Summary of design parameters for the alternative strategies.

Management Strategy	Design Storm focus at Site Scale	Design Storm focus at Watershed Scale	Other Unique Component
Traditional	2, 10	N/A	none
Traditional with Water Quality Component	2, 10	N/A	30-hour drawdown of WQ volume
Unit Release Rates	N/A	2, 25	release rate per acre to maintain condition at POI
Stratified Release Rate	N/A	2, 10, 25	release rate ratio based on contribution at POI
Low Impact Development	2, 10	N/A	Maintain runoff from LID storm (1-year)
Full Spectrum Detention	10, 25	N/A	72-hour drawdown of the EURV + 10%
Volumetric Design Procedure	N/A	2, 25	volume detention during critical time period

An observation related to design storms is the range analyzed for each strategy. Again referencing Table 4.1, it is found that each of the watershed based strategies design storms range from the 2- to the 25-year storms. It is noted that the more frequent storms

(< 2-year) are not explicitly analyzed for these strategies. With the site based strategies, design storms range from the 2- to the 10-year design storms, with the exception of the FSD strategy that ranges from the 10- to 25-year design storms. However, the unique components with the site based strategies also provide a focus on the smaller, more frequent storms.

4.1.2 Site runoff characteristics

The alternative strategies examined in this research have been discussed in previous chapters in regards to their goals, development and implementation requirements. A model scenario has been performed for each strategy that implemented the specified design requirements in each applicable area. Resulting hydrographs for each strategy and from each applicable area are provided in the Appendices as follows:

- Traditional - Appendix J
- Traditional with Water Quality Component - Appendix K
- Unit Release Rate - Appendix L
- Stratified Release Rate - Appendix M
- Low Impact Development - Appendix N
- Full Spectrum Detention - Appendix O
- Volumetric Design Procedure - Appendix P

Control structure design for each facility serving each applicable area was implemented in a manner to strictly meet the strategy objectives. For example, if no requirement was specified for the 25-year design storm, discharge was maximized immediately above the maximum water surface level resulting from the stage design for the 10-year storm. This approach is intended to give an accurate depicting of the strategy being used in practice.

It is observed from Table 4.1 and 4.2 that strategies that do not focus on "fringe" storms, fringe storms being the smallest or largest storms analyzed with this research, that post-development peak runoff rates exceed the predevelopment runoff peak runoff rates.

The exceptions are the VDP and URR strategy. These strategies, based on watershed analysis, detain the post-peak runoff rate to below the predevelopment peak runoff rate non-explicitly as a result of meeting the established discharge requirements for the 2-year storm. Also, although the Traditional with Water Quality and FSD strategy do not explicitly address the 1-year storm, each require an extended drawdown of an initial volume of runoff that perhaps unintentionally results in the post-development peak runoff rate being detained to below the predevelopment level.

Table 4.2 Comparison of percent differences in peak runoff rates when compared to predevelopment peak runoff rates.

Management Strategy	% difference compared to predevelopment peak runoff rate per design storm*			
	1-year	2-year	10-year	25-year
Traditional	+37	-7	-5	+20
Traditional w/ WQ	-22	-12	-3	+11
Unit Release rate	-41	-62	-8	-19
Stratified	+24	-17	-9	-16
Low Impact Development	-68	-36	-4	+11
Full Spectrum Detention	-93	-85	-23	-6
Volumetric Design Procedure	-29	-53	-30	-40

*** Based on an average from all applicable areas.**

Each management strategy also results in varied delays to the peak of the runoff hydrograph in the post-developed condition. It is observed in Table 4.3 that the most significant delays to peak runoff rate occur for smaller storms (1- and 2-year) and with the LID, URR and FSD strategies.

The following sections discuss runoff hydrograph characteristics resulting from each strategy. In all cases, runoff hydrographs are provided for the full range of design storms (1-, 2- , 10- and 25-year) analyzed with this research, for comparison purposes. Table 4.2 provides a summary of peak runoff rate comparisons to the predevelopment peak runoff rates. This Table and the characteristics described in this section will be referenced in later sections when examining results for the overall watershed.

Table 4.3 Average minutes that the post-development peak occurs after the predevelopment peak for each management strategy. Recall 3 minute time steps were used for model routing.

Management Strategy	Post-development peak runoff rate delay after predevelopment peak (minutes)			
	1-year	2-year	10-year	25-year
Traditional	5	7	4	0
Traditional w/ WQ	15	9	4	0
LID	66	21	9	6
URR	89	21	6	2
Stratified	6	8	5	3
FSD	163	123	8	3
VDP	14	24	7	7

4.1.2.1 Traditional

The Traditional strategy requires the post-peak runoff rate to be maintained below the pre-peak runoff rate for the 2- and 10-year design storms. The following characteristics are observed from the traditional strategy hydrographs:

A. Post-development peak runoff rates are maintained to below the predevelopment peak runoff rates for the 2- and 10-year design storms, per strategy design requirements.
B. Predevelopment peak runoff rates *are not* maintained for design storms outside the range of the required as part of the design strategy (the 1- and 25-year design storms). This is found to be the case for all applicable areas.
C. Runoff begins sooner with the post-development condition in all cases.
D. Post-development condition results in longer duration of higher flows when compared to predevelopment.
E. The post-development peak occurs slightly after the predevelopment peak in all cases, with the exception of the 25-year design storm that has no delay.

4.1.2.2 Traditional with Water Quality Component

The Traditional with Water Quality strategy requires the post-peak runoff rate to be maintained below the pre-peak runoff rate for the 2- and 10-year design storms. In addition, the strategy requires the 30-hour drawdown of the water quality volume. The following characteristics are observed from the Traditional with Water Quality strategy hydrographs:

A. Post-development peak runoff rates are maintained to below the predevelopment peak runoff rates for the 2- and 10-year design storms, per strategy design requirements. Also, as a result of the water quality volume drawdown requirement, the 1-year post-development peak runoff rate is also detained to below the predevelopment peak runoff rate.
B. The predevelopment peak runoff rate *is not* maintained for the 25-year design storm. This is the case for all applicable areas.
C. Measurable runoff begins later with the post-development condition in all cases. The ascending limb of the hydrograph is more vertical as the initial water quality volume is detained for the extended drawdown.
D. Post-development condition results in longer duration of higher flows when compared to predevelopment.
E. The post-development peak occurs slightly after the predevelopment peak for the 2- and 10-year design storms. A delay is more significant with the 1-year storm and the 25-year design storm that has no delay.

4.1.2.3 Unit Release Rates

The Unit Release Rate strategy requires the post-peak runoff rates to be maintained below release rates generated from the established unit release rate (cfs/acre) for the 2- and 25-year design storms. The following characteristics are observed from the Unit Release Rates strategy hydrographs:

A. Post-development peak runoff rates are maintained to below the predevelopment peak runoff rates for the full range of design storms with the exception of the 10-year storm from applicable area A.
B. Post-development condition results in longer duration of higher flows when compared to predevelopment. Lower flows subject to only the 2-year orifice are extended significantly longer when compared to the predevelopment condition.
C. The 2-year post-development peak is detained below the 1-year predevelopment peak runoff rate in all cases. In many cases, the 25-year post-development peak is detained below the 10-year predevelopment peak runoff rate.

D. The post-development peak occurs slightly after the predevelopment peak for the 10- and 25-year design storms. A delay is more significant with the smaller storms (1- and 2-year).

4.1.2.4 Stratified Release Rates

The Stratified Release Rate strategy requires the post-peak runoff rates to be maintained below allowable release rates generated from those established based on a watershed analysis that considered sub-basin contribution to the point of interest in the watershed for the 2-, 10- and 25-year design storms. The following characteristics are observed from the Stratified Release Rate strategy hydrographs:

A. Post-development peak runoff rates are maintained to below the predevelopment peak runoff rates for the 2-, 10- and 25-year design storms
B. Predevelopment peak runoff rates *are not* maintained for the 1-year design storm with the exception of applicable area C.
C. Runoff begins sooner with the post-development condition in all cases.
D. Post-development condition results in longer duration of higher flows when compared to predevelopment. Hydrograph receding limb shape is similar to the traditional strategy.
E. The post-development peak occurs slightly after the predevelopment peak in all cases.

4.1.2.5 Low Impact Development

The Low Impact Development strategy requires the post-peak runoff rate to be maintained below the pre-peak runoff rate for the 2- and 10-year design storms. In addition, the strategy requires the design to maintain the runoff hydrograph using a variety of design strategies to maintain the predevelopment hydrograph for the LID storm. For the purpose of model representation, the LID storm was determined to be the 1-year design storm. Retention was provided to maintain the LID design storm hydrograph volume, which also detained the peak to below the predevelopment peak in all cases. The time of concentrations in each applicable area is also maintained to the predevelopment condition. The following characteristics are observed from the Low Impact Development strategy hydrographs:

A. Post-development peak runoff rates are maintained to below the predevelopment peak runoff rates for the 1-, 2- and 10-year design storms. The peak for the smaller storms (1- and 2-year) is reduced to well below the predevelopment condition.
B. Predevelopment peak runoff rates *are not* maintained for the 25-year design storm with the exception of applicable area A.
C. Runoff begins later with the post-development condition in all cases. The ascending limb of the hydrograph is nearly vertical as the initial volume is retained on-site. With the smaller storms (1- and 2-year), runoff begins after the predevelopment peak.
D. Post-development condition results in longer duration of higher flows when compared to predevelopment. However, durations and magnitude of flows are not as pronounced on the receding limb as compared to other strategies due to the retained volume.
E. The post-development peak occurs slightly after the predevelopment peak for the 10- and 25-year design storms. A delay is more significant with the smaller storms (1- and 2-year).

4.1.2.6 Full Spectrum Detention

The Full Spectrum Detention strategy requires the post-peak runoff rate to be maintained below the pre-peak runoff rate for the 10- and 25-year design storms. In addition, the strategy requires the 72-hour drawdown of the excess urban runoff volume (EURV) plus 10%. The following characteristics are observed from the Full Spectrum Detention strategy hydrographs:

A. Post-development peak runoff rates are maintained to below the predevelopment peak runoff rates for the full range of design storms. Also, resulting from the extended drawdown of the EURV plus 10%, the smaller storms (1- and 2-year) are practically removed from the post-development runoff.
B. Similar to the LID strategy, runoff begins later with the post-development condition in all cases. With the smaller storms (1- and 2-year), runoff begins after the predevelopment peak.

C. Receding limbs for the smaller storms (1- and 2-year) are very low flow and drawn out for a duration well beyond the predevelopment peaks. The receding limb for post-development condition for larger storms results in longer duration of higher flows when compared to predevelopment. The receding limb for these larger storms is similar to the magnitude and shape of the Traditional strategy.
D. The post-development peak occurs significantly after the predevelopment peak for the 1- and 2-year design storms. A small delay occurs with the 10-year storm and the 25-year design storms.

4.1.2.7 Volumetric Design Procedure

The Volumetric Design Procedure strategy requires the post-development runoff volume to be maintained below the predevelopment runoff volume for the critical time period for the 2- and 25-year design storms. The following characteristics are observed from the Volumetric Design Procedure strategy hydrographs:

A. Post-development peak runoff rates are maintained to below the predevelopment peak runoff rates for the full range of design storms.
B. The 25-year post-development peak runoff rate is detained below the 10-year predevelopment peak runoff rate in all cases. With the exception of applicable area D, the 2-year post-development peak runoff rate is detained below the 1-year predevelopment peak runoff rate.
C. Post-development condition results in longer duration of higher flows when compared to predevelopment. Lower flows subject to only the 2-year orifice are extended significantly longer when compared to the predevelopment runoff hydrographs. Shape of the receding limb is similar to the unit release rate.
D. The post-development peak occurs slightly after the predevelopment peak for the 10- and 25-year design storms. A delay is more significant with the smaller storms (1- and 2-year).

4.2 Watershed Runoff Characteristics

Recall the point of interest (POI) as the point furthest downstream within the subject watershed. Runoff hydrographs resulting from the implementation of each alternative management strategy within the watershed are compared for each design storm (Appendix Q). Table 4.4 provides a summary of peak runoff rates, along with the percent difference compared to the peak runoff rates for the baseline condition simulation described in Section 3.2.3. Also included for comparison purposes are the results for the existing and existing with no detention simulations, also described in Section 3.2.3.

Table 4.4 Summary of peak runoff rates at the watershed outfall based for the alternative strategies. Results are compared to the baseline values established in Section 3.2.3. Highlighted values indicate where the resulting peak runoff rate is less than the baseline peak runoff rate.

Management Strategy	1-year		2-year		10-year		25-year	
	Outflow (cfs)	% diff.	Outflow (cfs)	% diff.	Outflow (cfs)	% diff.	Outflow (cfs)	% diff.
Traditional	170	8.2	276	9.3	495	-1.1	735	1.9
Traditional w/ WQ	162	2.8	241	-4.3	491	-1.9	735	1.9
LID	144	-8.6	249	-1.1	478	-4.5	739	2.5
URR	161	2.1	267	6.0	491	-2.1	698	-3.2
Stratified	178	13.1	281	11.6	511	2.1	669	-7.2
FSD	161	2.3	256	1.7	471	-5.9	678	-5.9
VDP	176	11.6	259	2.6	484	-3.4	665	-7.7
No Management	198	25.2	350	38.9	567	13.3	744	3.1
Existing*	187	18.4	285	13.2	527	5.3	706	-2.1
Baseline	158	0.0	252	0.0	501	0.0	721	0.0

*** Based on existing conditions within the watershed. Management would have been based on the traditional strategy. However, predevelopment conditions for the applicable areas would have varied based on the designer's analysis as opposed to the assumed, consistent predevelopment condition used for the purpose of this research.**

Table 4.4 demonstrates that maintaining the predevelopment peak runoff rate at the outfall of the watershed does not occur in most cases. No strategy achieves this goal for the full range of design storms. Results show there to be more success with maintaining the peak runoff rate to the below the target rate with the larger design storms (10- and 25-year) for most strategies. Only the LID strategy meets the watershed baseline target for the 1-year design storm. And only the LID and traditional with Water Quality strategies achieved the baseline goal for the 2-year design storm. It should also be noted that the URR unit release rate could be further reduced to at least meet the goal for the 2-year design storm. For the purpose of this research the unit release rates were developed by selecting a rounded release rate that most closely met the baseline target.

The following sections discuss the characteristics of the overall watershed that potentially affect the response seen with the implementation of the alternative management strategies. Each strategy is then considered individually in context with the results provided in Table 4.4.

4.2.1 Watershed characteristics

The subject watershed model used with this research is characterized as an urbanized area that has been mostly developed for many years, well prior to stormwater management regulation for development. This is typical across the country considering stormwater management regulation is relatively recent compared to the settlement of cities and towns. Development inherently increases runoff volumes and peak runoff rates. Although stormwater management can be designed to maintain predevelopment runoff rates at the site outfall, increases in runoff volumes as they convey throughout the watershed converge to cause increase in peak rates downstream, as previously described. As would be typical of the urban watersheds described above, there are aspects of these watersheds that may affect the downstream condition and may or may not alter the effects of a stormwater management strategy applied to development within the watershed. The following sub-sections discuss those components applicable to the subject watershed so that they may be considered when evaluating the results presented with this research.

4.2.1.1 Applicable Areas

Recall that an applicable area is defined for the purpose of this research as an area subject to stormwater management. As visually depicted in Appendix G, applicable areas only amount to a portion of the overall watershed. The total area contained within these applicable areas sums to 88.55 acres, approximately 23% of the subject watershed. Some previous studies that evaluate stormwater management at the watershed scale consider that all sub-basins are subjected to the management strategy. However, it is suggested that, except with newly developing watersheds, this may not be an accurate depiction for making this evaluation. Even then, scale of sub-basins should be carefully considered. Considering management strategies with management only applied to a coverage level that may be more practical could serve to determine the best strategy for evaluating effects at a point of interest downstream.

4.2.1.2 Critical locations

The critical location for the purpose of this research is the outfall of the subject watershed. The selection of this location is due to flooding issues immediately downstream. Results provide insight into what management strategy may provide for the best protection against flooding generated from the subject watershed. However, a watershed characteristic that it is suggested is typical of the urbanized watershed is the presence of regional stormwater facilities. These facilities are typically located and installed to mitigate for downstream issues caused by the increases in runoff associated with upstream development, often older development not provided stormwater management. These ponds typically handle flows from large drainage areas that include a significant portion of a watershed. It is typically not feasible to construct a pond treating large areas to meet specific site stormwater management requirements. Instead they are designed to provide the maximum detention possible based on available storage. The subject watershed includes two regional facilities (see Appendix G for location):

- Owens Park facility: This facility accepts flows from approximately 180 acres of the subject watershed (±46%). Three of the nine applicable areas are upstream of the Owens park facility. For the following reasons, it does not appear that the Owens pond would have a significant effect on comparison of watershed runoff characteristics produced by each strategy representation:
    - Considering the amount of flow through the pond and the minimum variations in the hydraulic grade line (HGL) within the pond for the management strategies (see Table 4.5), difference in flows released from the pond would be inconsequential.
    - A 30' weir with a crest elevation at 2094.00 feet and has potential for impacting discharge results. However, there is no case where the weir is not activated for the 10- and 25-year storm events. Therefore, there is not

a dramatic change in the stage-discharge capability of the pond's control structure. For all simulations, the range of the HGL is relatively small.

Table 4.5 Maximum HGL in the Owens Park regional facility with each management strategy implemented throughout the watershed.

Management Strategy	Maximum Hydraulic Grade Line in pond			
	1-year	2-year	10-year	25-year
Traditional	2,092.50	2,092.98	2,094.63	2,095.43
Traditional w/ WQ	2,092.39	2,092.83	2,094.62	2,095.45
URR	2,092.46	2,092.91	2,094.58	2,095.35
LID	2,092.37	2,092.82	2,094.52	2,095.38
Stratified	2,092.51	2,093.01	2,094.62	2,095.37
FSD	2,092.40	2,092.84	2,094.39	2,095.21
VDP	2,092.46	2,092.95	2,094.54	2,095.28
Existing	2,092.54	2,093.01	2,094.64	2,095.38
No management	2,092.70	2,093.30	2,095.04	2,095.73

- Lower Wong Park facility: This facility accepts flows from approximately 53 acres of the subject watershed (±13%). One of the nine applicable areas is upstream of the Wong Park facility. Similar to the Wong facility, it is noted from Table 4.6 that small variations occur with the HGL between all of the management strategy simulations. However, a 36" riser structure with the crest set at 2077.8 feet and a 30' weir with a crest set at elevation 2078.00 feet have potential for impacting discharge results. Table 4.6 shows that the riser structure is activated in all cases with the 2-year storm, although barely with the FSD strategy implemented in the watershed. The No Management, Traditional, Stratified and Existing Conditions scenarios do activate the 30' weir with the 2-year storm while the other strategy scenarios do not. However, in each case, the weir is barely activated (≤ 0.05 feet). Review of the stage-discharge curve (Figure 4.1) indicates the small rise in the HGL would result in minimal change in discharge. Therefore, similar to the Wong facility, the amount of flow through the pond and the minimum variations in the hydraulic grade line (HGL) within the pond for the management strategies result in minimal differences in flows released from the pond and could be inconsequential.

Table 4.6 Maximum HGL in the Lower Wong Park regional facility with each management strategy implemented throughout the watershed.

Management Strategy	Maximum Hydraulic Grade Line in pond			
	1-year	2-year	10-year	25-year
Traditional	2,077.45	2,078.05	2,078.39	2,078.59
Traditional w/ WQ	2,077.14	2,077.98	2,078.36	2,078.59
URR	2,077.17	2,077.93	2,078.35	2,078.54
LID	2,076.88	2,077.89	2,078.35	2,078.57
Stratified	2,077.44	2,078.03	2,078.38	2,078.60
FSD	2,076.91	2,077.81	2,078.35	2,078.57
VDP	2,077.21	2,077.96	2,078.35	2,078.52
Existing	2,077.47	2,078.04	2,078.40	2,078.50
No management	2,077.50	2,078.05	2,078.47	2,078.59

The discussion above is not intended to suggest that regional facilities do not impact the peak runoff rate at the watershed point of interest, but is only intended to illustrate that their effect does not prohibit a reasonable comparison of results between strategies evaluated in this research. In fact, the opposite appears to be true when considering the results of the Stratified strategy release rate generation which resulted in only those applicable areas subjected to regional facilities downstream to release 100% of the predevelopment peak runoff rate.

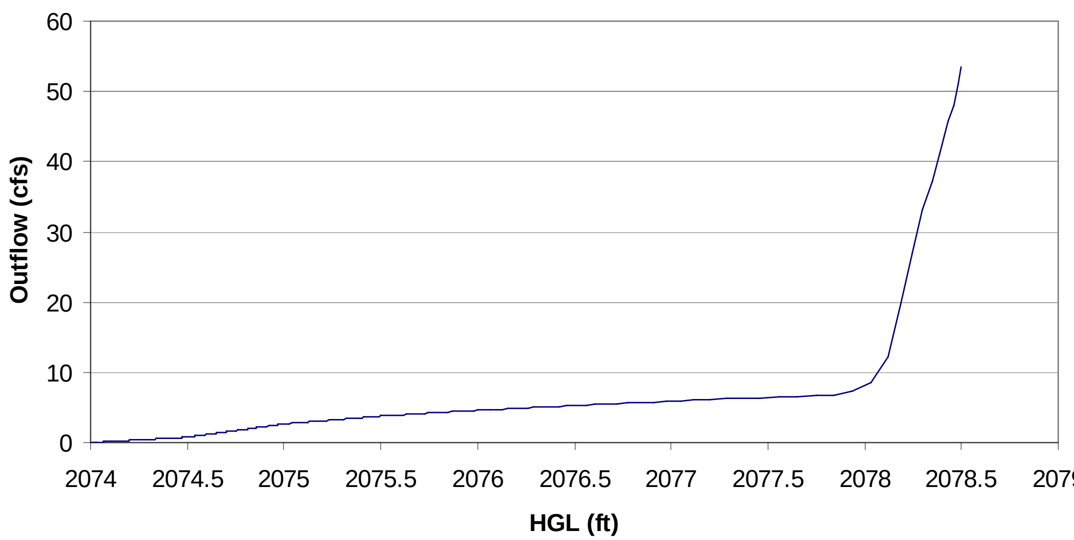


Figure 4.1 Stage-discharge curve for the Lower Wong park regional facility.

4.2.1.3 *Runoff Response Timing*

The occurrence of the peak runoff rate will occur sooner with a developed condition, as opposed to an undeveloped condition due to the increased velocities associated with impervious cover and channelized flows. For establishing the baseline targets for this research, the storm sewer system throughout the watershed is maintained except for within applicable areas. This characteristic would remain constant in an urbanized watershed despite the management strategy implemented. Therefore, peak timing considerations for the purpose of this research focus on the time of peak runoff generated from the applicable areas and the effects that may be seen downstream. This is further discussed for each management strategy in the next section.

4.2.2 Strategy correlation at a watershed scale

The observed characteristics in Section 4.1 for each of the management strategy's hydrograph from the applicable areas are considered as to how they influence the resulting hydrograph at the watershed outfall. The watershed characteristics are also considered in context with each strategy to evaluate:

- the potential influence of area subjected to stormwater management and
- the effects of regional facilities.

4.2.2.1 *Traditional*

A review of Table 4.7 shows that the Traditional strategy exceeds the baseline goal for the 1-, 2- and 25-year design storms. The strategy does meet the baseline goal for the 10-year design storm. Recall the design focus for the traditional strategy is peak reduction to the *site* predevelopment level for the 2- and 10-year design storms. This approach inherently disregards storm events outside of this range and can explain why the baseline target is not met for the 1- and 25-year design storms. This is further supported when referencing Table 4.2 that shows peak flows discharged at the site outfall greatly exceed the predevelopment peak from the site for these design storms. However, it is noted that the 2-year baseline is exceeded at the watershed outfall even with this predevelopment baseline met at the site outfall.

Table 4.7 Summary of peak runoff rates at the watershed outfall for the Traditional strategy. Results are compared to the baseline values established in Section 3.2.3.

Management Strategy	1-year		2-year		10-year		25-year	
	Outflow (cfs)	% diff.	Outflow (cfs)	% diff.	Outflow (cfs)	% diff.	Outflow (cfs)	% diff.
Traditional	170.25	8.2	275.6	9.3	495.2	-1.1	734.87	1.9
Baseline	157.72	N/A	252.05	N/A	500.69	N/A	721.00	N/A

This phenomenon is explained by the summation of many flat, longer duration hydrographs downstream within the watershed, as previously discussed in Chapters 1 and 2. From Table 4.3 it is found that the post-development peak runoff rates have a minimal delay in regards to time to peak rate occurrence. It is suggested that this coupled with the fact that the post-development peak runoff rates are maintained just below the predevelopment condition (see hydrographs - Appendix J) supports the phenomenon described above in that high flows are present downstream near the same time as discharge from other facilities in the watershed, since there is no lag in the discharge from the site.

4.2.2.2 Traditional with Water Quality Component

A review of Table 4.8 shows that the Traditional with Water Quality Component strategy exceeds the baseline goal for the 1- and 25-year design storms. The strategy does meet the baseline goal for the 2- and 10-year design storm. Recall the design focus for the Traditional with Water Quality strategy is peak reduction to the *site* predevelopment level for the 2- and 10-year design storms and a 30-hour drawdown of the water quality volume. This approach inherently disregards the 1- and 25-year storm event which can explain why these baseline targets are not met.

Table 4.8 Summary of peak runoff rates at the watershed outfall for the Traditional with a Water Quality component strategy. Results are compared to the baseline values established in Section 3.2.3.

Management Strategy	1-year		2-year		10-year		25-year	
	Outflow (cfs)	% diff.	Outflow (cfs)	% diff.	Outflow (cfs)	% diff.	Outflow (cfs)	% diff.
Traditional w/ WQ	162.18	2.8	241.12	-4.3	491.24	-1.9	735.1	1.9
Baseline	157.72	0	252.05	0	500.69	0	721	0

With the addition of the drawdown requirement for the water quality volume, it is noted that the percent difference of peak runoff rate compared to the baseline between the

Traditional and Traditional with Water Quality is reduced from 8.2% to 2.8%. From review of the runoff hydrographs in Appendix K, it is observed that the volume initially captured for the drawdown requirement results in this initial volume essentially being removed from runoff hydrographs. This prevents this volume from contributing to the peak runoff rate from the site. However, this volume "reduction" is not sufficient to meet the watershed baseline goal for the 1-year storm even though it results in a post-development peak runoff rate well below the predevelopment peak rate at the site scale (refer to Table 4.2). It is also inconsequential in regards to the 25-year storm for maintaining the predevelopment peak for this storm at the site scale (Table 4.2). This is because the water quality volume is small compared to the 25-year design storm. Peaks from this storm are still significant enough to prevent the strategy from meeting the watershed baseline.

Interestingly, the addition of the drawdown volume does result in the strategy meeting the 2-year watershed baseline goal. This result looks to be explained by the additional reduction in peak runoff rate at the site scale that results, on average, as a result of the drawdown requirement (reference Table 4.2). It does not appear that timing of runoff at the site scale is a factor since the peak runoff rate delay is nearly equal to the Traditional strategy (reference Table 4.3). In the case of this strategy, it appears that the reduction of peak flows from the site scale is enough to reduce the summation of hydrographs downstream for the 2-year storm.

4.2.2.3 Unit Release Rate

A review of Table 4.9 shows that the Unit Release Rate strategy exceeds the baseline goal for the 1- and 2-year design storms. The strategy does meet the baseline goal for the 10- and 25-year design storm. Recall the design focus for the URR strategy is based on a watershed analysis intended to maintain the peak at the watershed outfall to the predevelopment level for the 2- and 25-year design storms. This approach inherently disregards the 1- and 10-year storm event which can explain why the 1-year baseline goal is not met. However, design criteria ranging between the 2- and 25-year design storms perhaps inadvertently provides the ability to meet the 10-year watershed baseline and maintains post-development peak runoff rate to below predevelopment peak rates at the site scale (reference Table 4.2).

Table 4.9 Summary of peak runoff rates at the watershed outfall for the Unit Release Rate strategy. Results are compared to the baseline values established in Section 3.2.3.

Management Strategy	1-year		2-year		10-year		25-year	
	Outflow (cfs)	% diff.	Outflow (cfs)	% diff.	Outflow (cfs)	% diff.	Outflow (cfs)	% diff.
URR	161.08	2.1	267.22	6	490.69	-2.1	697.98	-3.2
Baseline	157.72	0	252.05	0	500.69	0	721	0

Despite the results in Table 4.4, this strategy could also be modified to meet the watershed baseline goal for the 2-year design storm. This would result in a lower unit release rate requirement for the 2-year storm and since the baseline is only exceeded by 2.1% for the 1-year storm, it is reasonable to assume that the baseline target could be met with all design storms with this strategy. However, the watershed analysis performed to establish the unit release rates only considered detention of flows within the applicable areas. Unless the goal is to retrofit existing facilities within an already built-out urban watershed, in practice, it would be difficult to perform this type of analysis in a non-urban watershed since location and extent of future development subject to stormwater management is not known.

From Table 4.2, it is observed that this strategy resulted in significant reduction of post-development peak runoff rates well below the predevelopment peak rates at the *site* scale. The greatest reductions are with the 1- and 2-year design storms (41 and 62% lower than pre-peaks, respectively). However, even with these significant reductions in peak runoff rates, the baseline targets for the watershed are not met with these two design storms. It is also noted from Table 4.3 that significant delay in peak flow also occurs at the site scale for this strategy. Referencing the site discharge hydrographs in Appendix L, it is observed that this strategy results in a hydrographs that indicates high flows for an extended period of time past the predevelopment peak along the receding limb. So although peaks are detained to well below predevelopment levels, these peaks last for an extended duration. It appears these peaks are high enough that as they are summed downstream, the summation is significant enough to prevent the baseline targets from being met with the smaller storms (1- and 2-year). The longer, drawn out flows along the receding limb appear to be most significant when the 2-year stage discharge is the only active discharge orifice. This could explain why the same occurrence does not happen in causing the larger storms not to meet the baseline objective.

The design requirements to meet the 2-year unit release rate inherently result in an extended drawdown of the 2-year design storm. Although this results in a significant peak reduction and peak delay at the site scale (Table 4.2 and 4.3), the benefit appears to be lost at the watershed scale. Considering the storage needed to detain these flows, this suggest that detention only to meet the objective may not be feasible for smaller storm events, especially when managed areas only make up a portion of the watershed.

4.2.2.4 Stratified Release Rates

A review of Table 4.10 shows that the Stratified Release Rate strategy exceeds the baseline goal for the 1- , 2- and 10-year design storms. The strategy does meet the baseline goal for the 25-year design storm. Recall the design focus for the Stratified Release Rate strategy is based on a watershed analysis that considers the flows contributing to the watershed outfall from sub-basins within the watershed. An allowable release rate is assigned based on the sub-basins contribution at the watershed POI for the 2-, 10- and 25-year design storms. This approach inherently disregards the 1-year storm event which can explain why the 1-year baseline goal is not met. This is further supported when reviewing Table 4.2 that shows the post-development peak runoff rate at the site scale exceeds the predevelopment rate by 24% for the 1-year storm.

Table 4.10 Summary of peak runoff rates at the watershed outfall for the Stratified Rate strategy. Results are compared to the baseline values established in Section 3.2.3.

	1-year		2-year		10-year		25-year	
Management Strategy	Outflow (cfs)	% diff.	Outflow (cfs)	% diff.	Outflow (cfs)	% diff.	Outflow (cfs)	% diff.
Stratified	178.39	13.1	281.36	11.6	511.16	2.1	669.05	-7.2
Baseline	157.72	0	252.05	0	500.69	0	721	0

The inability to meet the watershed objective for the 1-, 2- and 10-year design storms appears to be similar to the explanation for the Traditional strategy not meeting the baseline for specific storm events. Referring to the site runoff hydrographs for the Stratified strategy in Appendix M, it is observed that the general shape of the runoff hydrographs is similar to the Traditional strategy hydrographs. It is similarly found from Table 4.3 that the post-development peak runoff rates have a minimal delay in regards to time to peak rate occurrence. Although post-development peaks are reduced to below pre-development peaks at the site scale, the phenomenon of the summation of longer durations high flows downstream appears to occur within the watershed with the Stratified Release Rate strategy.

The watershed analysis performed to establish allowable release rates for sub-basins only considered the flows contributing from applicable areas. The Stratified strategy would typically develop a release rate map that dictates the allowable discharge for development in *all* sub-basins to meet a watershed baseline goal. Similar to the URR strategy that also required a watershed analysis, it is suggested that it would be difficult to meet the watershed goal in practice in an urbanized watershed since location and extent of future development subject to stormwater management is not known.

4.2.2.5 Low Impact Development

A review of Table 4.11 shows that the Low Impact Development strategy exceeds the baseline goal for the 25-year design storm. The strategy meets the baseline goal for the 1-, 2- and 10-year design storms. Recall the design focus for the LID strategy is peak reduction to the predevelopment level for the 2- and 10-year design storms. The strategy also requires maintaining the predevelopment peak and volume for the LID storm (determined to be 1-year design storm with this research). This approach inherently disregards the 25-year storm event which can explain why the 25-year watershed baseline goal is not met. This is further supported when reviewing Table 4.2 that shows the post-development peak runoff rate at the site scale exceeds the predevelopment rate by 11% for the 25-year storm.

Table 4.11 Summary of peak runoff rates at the watershed outfall for the Low Impact Development strategy. Results are compared to the baseline values established in Section 3.2.3.

Management Strategy	1-year		2-year		10-year		25-year	
	Outflow (cfs)	% diff.	Outflow (cfs)	% diff.	Outflow (cfs)	% diff.	Outflow (cfs)	% diff.
LID	144.17	-8.6	249.33	-1.1	478.39	-4.5	739.05	2.5
Baseline	157.72	0	252.05	0	500.69	0	721	0

This strategy's ability to meet the baseline target for the 1-year design storm is not surprising since the post-development runoff volume exceeding the predevelopment runoff volume for the 1-year design storm is retained on-site. Similar to the Traditional with Water Quality strategy, from a review of the runoff hydrographs in Appendix N, it is observed that the volume initially captured for the retention of volume exceeding the predevelopment volume for the 1-year storm results in this initial volume being removed from runoff hydrographs. This prevents this volume from contributing to the peak runoff rate from the site. However, unlike the Traditional with Water Quality strategy, this volume does appear to be significant enough to reduce peak flows and durations of those high flows enough to meet the baseline goals for the 1-, 2- and 10-year design storms. The reduction is not enough to meet the 25-year watershed baseline goal since the volume retained is small compared to the 25-year design storm's runoff volume. This again is further supported in Table 4.2 that shows peak runoff rate for the site discharge averages 11% higher than the predevelopment condition.

4.2.2.6 Full Spectrum Detention

A review of Table 4.12 shows that the Full Spectrum Detention strategy exceeds the baseline goal for the 1- and 2-year design storms. The strategy does meet the baseline goal for the 10- and 25-year design storm. Recall the design focus for the Full Spectrum Detention strategy is peak reduction to the predevelopment level for the 2- and 25-year design storms and a 72-hour drawdown of the EURV + 10%. This approach inherently disregards the 1- and 10-year storm event which can explain why these baseline targets are not met. However, a design criteria range between the 2- and 25-year design storms inadvertently provides the ability to meet the 10-year watershed baseline and maintains post-development peak runoff rate to below predevelopment peak rates at the site scale for all design storms (reference Table 4.2).

Table 4.12 Summary of peak runoff rates at the watershed outfall for the Full Spectrum Detention strategy. Results are compared to the baseline values established in Section 3.2.3.

Management Strategy	1-year		2-year		10-year		25-year	
	Outflow (cfs)	% diff.	Outflow (cfs)	% diff.	Outflow (cfs)	% diff.	Outflow (cfs)	% diff.
FSD	161.33	2.3	256.27	1.7	471.06	-5.9	678.16	-5.9
Baseline	157.72	0	252.05	0	500.69	0	721	0

A review of the site runoff hydrographs for this strategy in Appendix O reveals that the 72-hour drawdown of the EURV + 10% results in runoff hydrographs for the 1- and 2-year post-development storms that are virtually non-existent. Interestingly though, the baseline target is not met for these two storms. With this, it appears that the portions of the watershed not subjected to stormwater management are the flows that play the significant role for generation of the peak runoff rates at the watershed outfall. This supports the previous suggestion from Section 4.2.2.3 that the use of detention only to meet the baseline objective may not be feasible for smaller storm events, especially when managed areas only make up a portion of the watershed. However, the reduction in volume from the 72-hour drawdown does appear to remove enough volume from the peak of the runoff hydrograph to meet the watershed goals for the larger storm events (10- and 25-year).

4.2.2.7 Volumetric Design Procedure

A review of Table 4.13 shows that the Volumetric Design Procedure strategy exceeds the baseline goal for the 1- and 2-year design storms. The strategy meets the baseline goal for the 10- and 25-year design storms. Recall the design focus for the Volumetric Design Procedure strategy is based on a watershed analysis that requires post-development runoff volume be maintained to the predevelopment volume for the critical time period for the 2- and 25-year design storms. This approach inherently disregards the 1- and 10-year storm event which can explain why the 1-year baseline goal is not met. However, a design criteria range between the 2- and 25-year design storms inadvertently provides the ability to meet the 10-year watershed baseline and maintains post-development peak runoff rate to below predevelopment peak rates at the site scale for this storm (reference Table 4.2).

Table 4.13 Summary of peak runoff rates at the watershed outfall for the Volumetric Design Procedure strategy. Results are compared to the baseline values established in Section 3.2.3.

	1-year		2-year		10-year		25-year	
Management Strategy	Outflow (cfs)	% diff.	Outflow (cfs)	% diff.	Outflow (cfs)	% diff.	Outflow (cfs)	% diff.
VDP	176.00	11.6	258.64	2.6	484.08	-3.4	665.37	-7.7
Baseline	157.72	0	252.05	0	500.69	0	721	0

From Table 4.2, it is observed that this strategy resulted in significant reduction of post-development peak runoff rates well below the predevelopment peak rates at the site scale. The reductions with the 1- and 2-year design storms are 29 and 53% lower than pre-peaks, respectively. However, even with these significant reductions in peak runoff rates, the baseline targets for the watershed are not met for these two design storms. It is also noted from Table 4.3 that significant delay in peak flow also occurs at the site scale for this strategy for the 1- and 2-year design storms. Referencing the site discharge hydrographs in Appendix P, it is observed that this strategy results in a hydrographs that indicates high flows for an extended period of time past the predevelopment peak along the receding limb. So similar to the URR strategy, although peaks are detained to well below predevelopment levels, these peaks last for an extended duration. It appears these peaks are high enough that as they are summed downstream. The summation is significant enough to prevent the baseline targets from being met with the smaller storms (1- and 2-year). Also similar to the URR hydrographs, the longer and more drawn out flows along the receding limb appear to be most significant when the 2-year stage discharge is the only active discharge orifice. Again, similar to the URR, this could explain why the same occurrence does not happen in causing the larger storms not to meet the baseline objective.

4.2.2.8 Supplementary Model Scenarios

In addition to model simulations that implemented each of the management strategies evaluated in this research within the subject watershed, the following simulations, as described in more detail in Section 3.2.3, are provided for comparison purposes:

- Existing conditions scenario - Represents the conditions as they exist in the subject watershed based on field survey and as-built record drawings. All stormwater management in the watershed would have been subject to the Traditional strategy at the time of development.
- No management - Represents the existing condition of the subject watershed with all stormwater management facilities removed.

A review of Table 4.4 shows that the Existing Conditions scenario exceeds the baseline goal for the 1-, 2- and 10-year design storms. The strategy meets the baseline goal for the 25-year design storm. It is not practical to compare this model simulation to the baseline goals since it is unknown what predevelopment condition was considered for each applicable area. However, if the goal of stormwater management is to meet a downstream objective, it is suggested that a uniform predevelopment condition should be applied for development within the watershed. Support for this suggestion can be gained by observing the watershed outfall hydrographs in Appendix Q. The hydrographs show the existing condition peak runoff rate to exceed the peak rate of all other the other modeled scenarios, except for the No Management scenario for the 1-, 2- and 10-year design storm.

Results in Table 4.4 for the No Management scenario are as would be expected in that peak runoff rate exceeds the peak runoff rates generated by all other scenarios with the exception of the Traditional with Water Quality strategy for the 25-year storm. In fact the peak runoff rate for the 25-year storm is comparable to the three strategies that do not have specific design requirements for the 25-year storm: Traditional, traditional with Water Quality and LID. Also, considering the significantly higher peak runoff rates with smaller storms (1- and 2-year), when compared to all of the other strategy results, it is clear that the implementation of stormwater management does mitigate for increases in peak runoff rates downstream to some extent.

4.2.3 Other considerations

The previous sections in this chapter have focused on the runoff characteristics at a site and watershed scale. However, the implications of implementing the management strategies should also be considered. This section highlights the general implications so that they may be considered as part of a tailored stormwater management strategy in the next Chapter.

4.2.3.1 Volume requirements at the site scale

Depending on the management strategy, specific design requirement must be implemented to meet the strategy's objectives. For all of the strategies presented in this research, a detention facility would typically be used and a discharge control structure design to allow only the allowable discharge per the management strategy specifications for specified design storms. The exception is the LID strategy which would also required the retention of a specified volume on site.

Table 4.14 provides a summary of volumes required in the stormwater management facility at each applicable area for each management strategy to meet the strategies design specifications. Table 4.15 summarizes these values as a percent of the volume required compared to the Traditional strategy that required the least amount of storage volume in all cases. Recall that for the LID strategy, retention storage was provided to meet the volume reduction requirement associated with the strategy. However, even with this, the LID strategy still requires less on-site retention/detention storage than the FSD and VDP strategies. The 72-hour drawdown of the EURV + 10% requires significant storage to detain flows that are slowly released over a long duration. This long, extended drawdown results in the need for the largest available storage of all the strategies. From review of the control structure information for the VDP (reference Appendix P), it appears that the secondary discharge outlet, geared to maintain post-development runoff volumes to below predevelopment volumes during the critical time period results in the large storage requirements with this strategy.

Table 4.14 Maximum volume used in management facilities serving each applicable area for each management strategy.

Applicable Area	Volume used for the 25-year storm (ft^3)						
	Traditional	Traditional w/ WQ	LID	URR	Stratified	FSD	VDP
A	83,411	86,228	109,718	95,359	86,228	120,740	110,228
B	14,207	16,380	22,244	17,470	15,082	31,414	25,783
C	24,140	31,620	35,042	33,043	31,473	41,119	38,700
D	13,830	16,976	21,112	21,829	15,996	22,993	21,759
E	28,962	30,032	40,985	41,035	38,414	59,717	52,921
F	10,548	12,586	13,865	16,495	13,650	17,992	16,356
G	9,693	11,783	13,822	12,714	11,139	16,754	14,934
H	8,921	11,779	13,600	12,357	10,857	16,352	15,163
I	34,726	43,161	46,910	44,988	36,404	45,529	45,512

Recall from Section 4.2.2.3 that the URR 2-year unit release rate generally results in the extended detention of the 2-year design storm. With this, significant storage is required to detain the 2-year volumes. With this, storage requirements are found to be similar to the LID strategy. The Stratified and Traditional with Water Quality require the lowest storage volume, with the exception of the Traditional strategy.

Table 4.15 Percent of volume used in management facilities serving each applicable area for each management strategy compared to volume used with the implementation of the Traditional strategy. Red = high storage, yellow = mid-range and green = low storage.

Applicable Area	% of volume used compared to volume used with the implementation of the Traditional strategy						
	Traditional	Traditional w/ WQ	LID	URR	Stratified	FSD	VDP
A	100	103	132	114	103	145	132
B	100	115	157	123	106	221	181
C	100	131	145	137	130	170	160
D	100	123	153	158	116	166	157
E	100	104	142	142	133	206	183
F	100	119	131	156	129	171	155
G	100	122	143	131	115	173	154
H	100	132	152	139	122	183	170
I	100	124	135	130	105	131	131
Average ->	100	119	143	137	118	174	158

Relative cost estimates are provided in Table 4.16. These are based on the following equations from Brown and Schueler (1997):

$$C = 7.47V^{0.780}, \textit{ for detention (\$)} \tag{4.1}$$

$$C = 12.4V^{0.760}, \textit{ or extended detention (\$)} \tag{4.2}$$

Where V = volume, ft^3

Equation 4.1 is used to calculate cost for the Traditional and Stratified strategies and as part of the LID cost calculations. Equation 4.2 is used for the remaining strategies that would all result in ponds similar to those defined as extended detention. In addition to Equations 4.1, infiltration is considered with the LID strategy. Required volume reduction is assumed to be addressed with infiltration for the purpose of the cost analysis at \$2.25/ft^3 (EPA, 1999). A detention cost is also included for peak reduction requirements. Table 4.17 summarizes these costs as a percent of the cost required for implementation of the Traditional strategy that resulted in the least cost in all cases. It is noted that the LID strategy resulted in the highest costs as a result of the volume required to be removed via infiltration. Since LID can potentially include a wide variety of BMPs to achieve volume reduction, this number could vary greatly as is intended as a relative value only.

Table 4.16 Relative capital cost estimates for implementing each strategy at each applicable area.

Applicable Area	Estimated Costs ($)						
	Traditional	Traditional w/ WQ	LID	URR	Stratified	FSD	VDP
A	51,508	69,906	139,171	75,463	52,860	90,288	84,249
B	12,950	19,783	39,653	20,776	18,580	32,452	27,928
C	19,582	32,613	57,069	33,723	32,498	39,819	38,026
D	12,681	20,328	35,501	24,609	19,430	25,600	24,549
E	22,571	31,361	57,810	39,757	37,812	52,876	48,237
F	10,266	16,193	27,789	19,889	17,224	21,246	19,761
G	9,611	15,402	27,691	16,318	14,758	20,126	18,441
H	9,008	15,398	28,682	15,969	14,473	19,758	18,656
I	26,003	41,313	74,299	42,636	36,299	43,025	43,013

Table 4.17 Percent of cost for management facilities serving each applicable area for each management strategy compared to cost for the implementation of the Traditional strategy. Red = high cost, yellow = mid-range and green = low cost.

Applicable Area	% of cost compared to cost with the implementation of the Traditional strategy						
	Traditional	Traditional w/ WQ	LID	URR	Stratified	FSD	VDP
A	100	136	270	147	103	175	164
B	100	153	306	160	143	251	216
C	100	167	291	172	166	203	194
D	100	160	280	194	153	202	194
E	100	139	256	176	168	234	214
F	100	158	271	194	168	207	192
G	100	160	288	170	154	209	192
H	100	171	318	177	161	219	207
I	100	159	286	164	140	165	165
Average ->	100	156	285	173	150	207	193

4.2.3.2 Implementation constraints

There are three general constraints that can serve as an impediment towards implementation of the management strategies presented in this report:

1. Physical constraints
    a. LID depends on a reduction of volume from the runoff hydrograph. This is typically achieved by utilizing a number of design techniques in unison, but almost always has a heavy infiltration component. Considering infiltration, soils may be a constraint if permeability rates are not adequate.
    b. LID site design (foot printing to reduce the CN value, etc) or strategies that require large storage requirements may not be physically practical in urbanized areas where site projects may have limited area as with infill projects and redevelopment projects.
2. Practical planning constraints with watershed analysis - As mentioned in Section 4.2.2 in regards to the URR and Stratified strategies, specifying unit or allowable release rates presents a challenge since it is not known where future management will exist and where it will not exist in an already heavily developed watershed.

Trying to meet watershed baseline objectives may not be possible when strategies are only implemented in applicable areas.

3. Cost/benefit - Cost should be considered at the site scale as related to storage requirements and at the watershed scale when justifying a management strategy that requires a watershed analysis.

5 <u>Tailored Stormwater Management</u>

Chapter 5 provides an evaluation of the alternative management strategies in regards to their potential benefit with implementation in the subject watershed. The evaluation leads to the selection of a base strategy determined to best serve the subject watershed and ultimately modifications to that strategy that result in a tailored management strategy. This selected and modified strategy is defined as the "tailored" stormwater management strategy for the specified watershed.

5.1 Management Strategy Evaluation, Selection and Modification

Results provide several metrics to consider for selection of the optimal management strategy for implementation in the subject watershed. These are specified as:

1. The ability to meet the baseline goals, or provide optimal reduction in peak runoff rate at the watershed POI for the range of design storms,
2. The ability to maintain the predevelopment runoff rate at the site scale for the range of design storms,
3. The relative volume required to implement the strategy at the site scale.
4. The relative cost to implement the strategy at the site scale.

In addition, the three major shortcomings cited by Emerson et al. (2004) are also considered. Recall these shortcomings as:

a. Although peak runoff rates are detained to below predevelopment rates at the site, streambank erosion is increased with increased duration and volume of runoff.
b. The design of detention ponds are usually focused on specified return interval storms, such as the 2-year or 100-year storms, that account for only a small fraction of yearly precipitation.
c. Since detention ponds are designed on a site-by-site basis, their watershed-wide performance can not be assured.

5.1.1 Results evaluation

From Table 4.4, recall that none of the alternative strategies achieved the baseline goal for all design storms. A look at this table in context to #1 and #2 above results in the following points:

A. Those strategies that do not specifically require reduction of the 25-year design storm to predevelopment peak runoff rates at the site scale (Traditional, Traditional with Water Quality and LID) do not accomplish this at the site scale nor do they meet the watershed baseline goal for the 25-year design storm. All strategies that had specific design criteria for the 25-year design storm met the watershed baseline goal for the storm event.
B. Only the Stratified strategy did not meet the watershed baseline goal for the 10-year design storm. Although this strategy reduces the post-development runoff peak to below the predevelopment peak at the site scale for this design storm (reference Table 4.2), it is suggested that the strategy cannot meet the watershed goals since the method for assigning release rates is based on sub-basin contribution at the watershed POI and many sub-sheds within the watershed are not subject to stormwater management. Therefore, their contributions are left unmanaged.
C. The Stratified strategy exceeds the baseline peak runoff rates by a greater percentage than all other alternative strategies for the 1-, 2- and 10-year design storms.
D. The Traditional strategy is the only strategy that met the watershed baseline goal for only one design storm (the 10-year design storm). Excess above the baseline peak runoff rates ranked third highest, after the Stratified and VDP strategies for the 1- and 2-year design storms.
E. Only the LID strategy meets the goal for the 1-year design storm. And only the LID and Traditional with Water Quality met the goal for the 2-year design storm. It is noted that the FSD strategy required a significantly larger volume and drawdown with the EURV + 10% when compared to the 30-hour drawdown of the water quality volume associated with the traditional with Water Quality strategy. However, this larger volume and drawdown, although resulting in lower

peak runoff rates at the site scale (refer to Table 4.2), does not result in significantly different peak rate reductions at the watershed POI for the 1-year design storm. In addition, the FSD results in a higher peak runoff rate than the Traditional with Water Quality at the watershed POI with the 2-year design storm. This suggests that extending the drawdown and increasing the volume for the drawdown of an initial runoff volume does not imply better results downstream.

F. The VDP is second only to the Stratified strategy in regards to the percent difference above the baseline peak runoff rate for the 1-year design storm. The VDP also does not meet the watershed baseline for the 2-year storm even though there is a great decrease in the peak runoff rate at the site scale (reference Table 4.2). This suggests that a reduction below predevelopment peak rate to this level (> 30% for 1-year and 50% for 2-year) at the site scale is not warranted.

G. The FSD and the URR do not meet the watershed baseline for the 1- and 2-year design storms even though the site release rates are reduced to well below the predevelopment peak runoff rates (Table 4.2). This suggests that a reduction below predevelopment rate to this level (> 40% for 1-year and 60% for 2-year) at the site scale is not warranted.

From Tables 4.15 and 4.17, relative volumes required for implementation of each strategy and associated costs are provided. Volumes are used to generate construction costs. However, the additional cost for loss of land area needed to provide the required volume is also considered. A look at these tables in context to #3 and #4 above results in the following items:

H. The FSD and the VDP result in the largest storage volume requirements, on average. This also translates to the highest construction cost increase compared to the Traditional strategy (nearly 100%), except for the LID strategy, which shows nearly a 200% increase.

I. The Traditional with Water Quality and Stratified result in the smallest increase (± 20%) when compared to the Traditional strategy. Costs increase compared to the Traditional strategy are also the smallest for these two strategies, resulting in an increase near 50%.

J. The LID and URR storage volume requirements are mid-range in comparison with the alternative strategies, however while the URR costs are also mid range (70% increase compared to Traditional), the LID cost as the highest of all strategies, as previously mentioned. The high costs associated with LID stem from the costs of BMPs more complex than detention ponds related to the volume reduction requirements of the strategy.

5.1.2 Evaluation of Emerson's shortcomings

The Virginia Erosion and Sediment Control Handbook (1992) states that studies have shown that most natural streams are formed with a bank-full capacity to pass runoff from a storm with a 1.5- to 2-year recurrence interval. Emerson's first shortcoming states that: "although peak runoff rates are detained to below predevelopment rates at the site, streambank erosion is increased with increased duration and volume of runoff." This is found to be the case downstream of the subject watershed at the urban/rural interface downstream where piped flows discharge to the open channel. This shortcoming may be partially addressed with the following:

- Duration of bank-full conditions can be maintained to the predeveloped, or baseline condition. This assumes that although duration of flows higher than the predevelopment exists, this would occur along the receding limb of the hydrograph, after the peak (bank-full) has occurred for the 1.5- to 2-year design storm.
- Excess volumes that are inevitable in urbanized watersheds can pass after the peak (bank-full) occurs with the 1.5- and 2-year recurrence intervals.

The two items are conceptual in nature and intended to address Emerson's concern by maintaining the level in which natural streams are formed. This concept is illustrated in Figure 5.1 using the portion of the watershed POI hydrographs that would represent bank-full, or near bank-full conditions with the 2-year design storm. The optimal strategy would do the following to address Emerson's first shortcoming:

✓ Maintain the predevelopment peak for the 1.5- to 2-year design storm.

✓ Maintain the predevelopment duration and volume of runoff in the downstream channel at, and near, bank-full conditions.

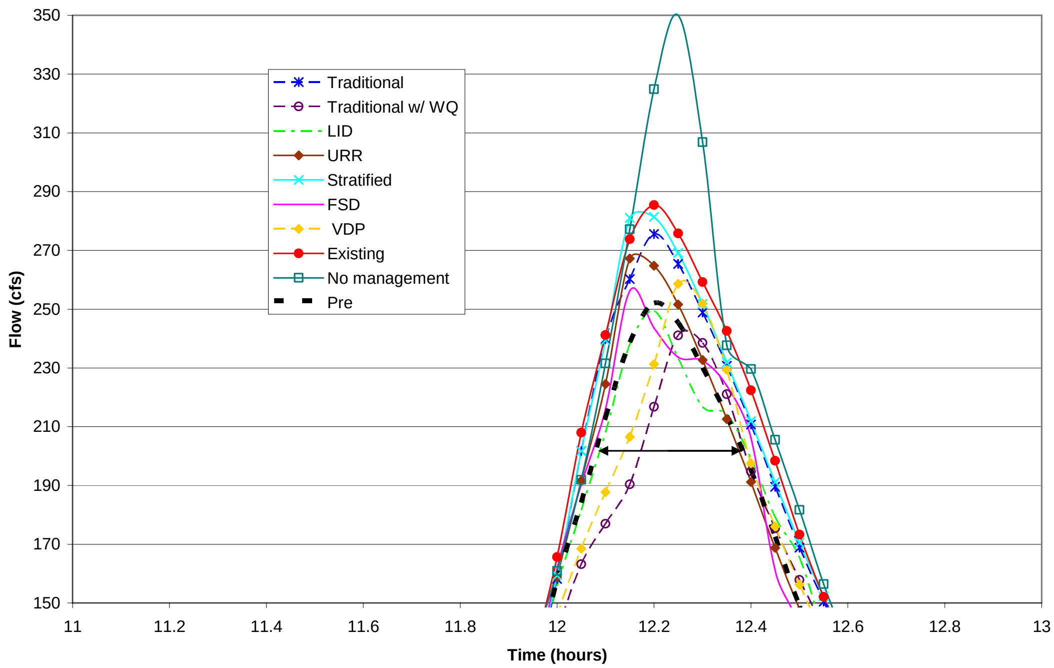


Figure 5.1 A portion of the watershed POI 2-year design storm hydrographs at, and near, the bank full condition. Note that arrows are provided to illustrate the concept of the duration of the high flows contributing to near bank full conditions for the predevelopment baseline condition.

It is not feasible to maintain the predevelopment volume for the entire duration of the runoff hydrograph in the urbanized watershed. However, it may be possible to maintain the volume during the high flow stage of the hydrograph. It is suggested that this could reduce the erosive impact on the downstream receiving stream. To attempt to satisfy this first of Emerson's shortcomings, the 1- and 2-year watershed POI hydrographs (Appendix Q) are evaluated for high flow duration. For example, this evaluation would note from Figure 5.1 that the LID and Traditional with Water Quality strategies meet the peak runoff rate baseline and also appear to maintain the duration, and therefore the volume during the high flow portion of the hydrograph, when compared to the predevelopment condition. Similar observation is made with the 1-year design storm (reference Appendix Q), except the Traditional with Water Quality strategy slightly exceeds the predevelopment peak with this storm. Recall that the predevelopment baseline for the purpose of this research is not the condition in its natural state, but rather a baseline target based on a point in time. It is not intended to be suggested that the natural state at the watershed POI could be returned to its natural condition in an urban watershed, but only that erosive conditions could be maintained to the levels that occurred with the at the baseline condition.

Emerson's second shortcoming related to the focus on specified return intervals, such as the 2-year and 100-year storms, has been discussed within this research in regards to evaluation of each of the alternative strategies. Since most annual precipitation events result in storms less than the 2-year design storm, the 1-year storm has also been included in the analysis presented in this paper. Emerson's third shortcoming has inherently been addressed with the evaluation of the performance of each of the management at the watershed scale.

5.1.3 Strategy Selection

The desired result is to select a strategy that meets the watershed baseline goal for the range of design storms. Since none of the strategies meet this goal, a strategy is selected that appears to provide the best results based on the analysis described in this report. The following key characteristics of the alternative strategies within the watershed resulted from the evaluation described in previous sections:

- Only strategies that have a specific management requirement for the 25-year design storm meet the baseline watershed goal for this storm event.
- The Traditional, Stratified, URR and VDP do not appear to have the potential to meet watershed baseline goals for the full range of design storms, mostly due to their inability to meet the watershed baseline goals for the smaller storms (1- and 2-year).
- Only the LID, FSD and the Traditional with Water Quality provide the best results for achieving, or coming close to achieving, the watershed baseline goals for the smaller storm events (1- and 2-year). However, the large volume (EURV + 10%) and long drawdown associated with the FSD do not provide better downstream results.
- The FSD and VDP result in large site storage requirements, resulting in relatively high construction costs.
- LID results in high construction costs.

- The Traditional, Traditional with Water Quality and Stratified result in the lowest site storage requirements and smallest construction costs, when compared to other strategies.
- Only the Traditional with Water Quality and LID strategies appear to maintain predevelopment duration of high flows (near bank-full) at the watershed POI. Therefore, these strategies provide the best possibility of addressing Emerson's first shortcoming regarding duration and volume of flows causing erosion.

Based on these generalized items resulting from evaluation, the Traditional, URR, Stratified and VDP strategies are removed from consideration as the optimal strategy for the subject watershed. The FSD is also removed since it is found that the larger site storage requirements for the 72-hour drawdown due not provide increased benefit at the watershed POI. The remaining strategies are the Traditional with Water Quality and LID. Although LID performs well for the range of design storms, the physical characteristics prohibit the ability to implement this strategy fully to its design specifications. These characteristics are the clayey soils and the urbanized nature of the watershed in that area on redevelopment and infill projects is limited, making the installation of integrated BMPs difficult to achieve. With this, the Traditional with Water Quality strategy is selected as the base strategy for implementation within the watershed.

5.1.4 Tailored Management for the Subject Watershed

With the selection of the Traditional with Water Quality strategy, it is noted that Emerson's second shortcoming regarding the range of storms analyzed is not fully addressed since the 1- and 25-year watershed baseline is not met. With the 1-year storm, this baseline is not met even though the strategy results in post-development peak runoff rates that average 22% less than the predevelopment peak runoff rates at the site scale. The URR, FSD and VDP all exceeded this average reduction, as much as almost 5 times in the case of FSD, yet none of these three met the watershed baseline goal. Only LID, which required on-site retention of the difference of volumes produced between the post- and predevelopment runoff for the 1-year design storm met the 1-year watershed baseline.

Inherently, Emerson's shortcoming regarding the full range of design storms is addressed since an analysis of performance at a watershed scale is presented. However, to attempt to address this more explicitly and provide a modified strategy that meets the baseline goal for the full range of design storms, a runoff retention component, similar to the LID strategy, was added to the Traditional with Water Quality strategy. An iterative process was performed that added an increasingly higher volume retention requirement. It was found that the full range of baseline goals for the watershed could not be met until the volume retained on site approached the difference of volumes produced between the post- and predevelopment runoff for the 1-year design storm, identical to the LID strategy requirement. Therefore, this modification is not feasible as a component to a tailored strategy for the same reasons the LID strategy is not applicable within the watershed. Recall that the Traditional with Water Quality strategy already produced post-peak runoff rates less than the predevelopment peak rates at the site scale. Since volume reduction to meet the baseline for the full range of design storms, and peak reduction for the 1-year storm at the site scale does not meet the watershed baseline goal, the percent difference in peak increase from the baseline peak of only 2.8% for the 1-year storm is deemed acceptable.

Although the 1-year watershed baseline goal did not appear to be obtainable using detention or feasible with retention of volumes, Emerson's shortcoming is partially addressed with a modification to the Traditional with Water Quality strategy that includes a requirement for the post-development peak runoff rate for the 25-year design storm be reduced to below the predevelopment peak runoff rate at the site scale. Recall from Table 4.2 that this strategy resulted in a post-development peak runoff rate that averaged 11% above the predevelopment peak runoff rate at the site scale for the 25-year design storm. Also recall that all of the strategies that did specifically address the 25-year design storm did meet the watershed baseline. To evaluate the strategy with the modified component, the control structure serving each applicable area was modified to maintain the post-development peak runoff rate to the predevelopment level for the 25-year storm (Appendix R). The results for the full range of design storms with implementation of this tailored strategy are found in Appendix S. Note from Tables 5.1 and 5.2 that the observation noted in the review of strategies that described the 25-year watershed baseline goal being met when specifically addressed by the strategy requirements, hold true with the tailored strategy.

Table 5.1 Summary of peak runoff rates at the watershed outfall for the Tailored strategy. Results are compared to the baseline values established in Section 3.2.3.

Management Strategy	1-year		2-year		10-year		25-year	
	Outflow (cfs)	% diff.	Outflow (cfs)	% diff.	Outflow (cfs)	% diff.	Outflow (cfs)	% diff.
Tailored	162.18	+2.8	241.12	-4.3	491.24	-1.9	695.18	-3.6
Baseline	157.72	N/A	252.05	N/A	500.69	N/A	721.00	N/A

Table 5.2 Summary of stages at the watershed outfall for the Tailored strategy. Results are compared to the baseline values.

Management Strategy	Stage (ft)			
	1-year	2-year	10-year	25-year
Tailored	2059.28	2059.77	2061.07	2061.75
Baseline	2059.24	2059.81	2061.16	2061.94

With the addition of the modification to the Traditional with Water Quality strategy to include site peak reduction requirements, the volume requirement and construction costs were evaluated for comparison to all strategies (see Tables 5.32 through 5.6). It was found that this Tailored strategy resulted in volume requirements and construction costs in the mid range, when compared to all strategies (reference Tables 4.14, 4.15, 4.16 and 4.17). On average the modification resulted in a 12% increase in storage volume required when compared to the Traditional with Water Quality strategy with no modification. The average increase in cost when compared to the Traditional with Water Quality strategy approached 9%.

Table 5.3 Maximum volume used in management facilities serving each applicable area for Tailored, Traditional with WQ, or Traditional strategy.

Applicable Area	Volume used for the 25-year storm		
	Traditional	Traditional w/ WQ	Tailored
A	83,411	86,228	105,309
B	14,207	16,380	16,564
C	24,140	31,620	33,949
D	13,830	16,976	20,238
E	28,962	30,032	36,077
F	10,548	12,586	14,621
G	9,693	11,783	13,449
H	8,921	11,779	12,035
I	34,726	43,161	44,827

Table 5.4 Percent of volume used in management facilities serving each applicable area for Tailored, Traditional with WQ, or Traditional strategy, compared to volume used with the implementation of the Traditional strategy.

Applicable Area	Traditional	Traditional w/ WQ	Tailored
A	100	103	126
B	100	115	117
C	100	131	141
D	100	123	146
E	100	104	125
F	100	119	139
G	100	122	139
H	100	132	135
I	100	124	129
Average ->		119	133

Table 5.5 Relative capital cost estimates for implementing the Tailored, Traditional, or Traditional strategy.

Applicable Area	Estimated Costs ($)		
	Traditional	Traditional w/ WQ	Tailored
A	51,508	69,906	81,376
B	12,950	19,783	19,952
C	19,582	32,613	34,423
D	12,681	20,328	23,233
E	22,571	31,361	36,051
F	10,266	16,193	18,147
G	9,611	15,402	17,031
H	9,008	15,398	15,651
I	26,003	41,313	42,520

Table 5.6 Percent of cost for management facilities serving each applicable area for the Tailored, Traditional, or Traditional strategy compared to cost for the implementation of the Traditional strategy.

Applicable Area	Traditional	Traditional w/ WQ	Tailored
A	100	136	158
B	100	153	154
C	100	167	176
D	100	160	183
E	100	139	160
F	100	158	177
G	100	160	177
H	100	171	174
I	100	159	164
Average ->		156	169

5.2 Tailored Management Guidance

Recall that "tailored" stormwater management refers to the selection of an optimal management strategy, with possible modifications, to be implemented within a specific watershed. It is suggested that this selection should be based from the results of a watershed analysis that examines results at a point, or points, of interest within the watershed. The watershed analysis should evaluate available alternative strategies for a range of design storms. An analysis similar to the one presented here would include the following steps:

1. Identify a point, or points, of interest that experiences, or has the potential to experience with future development, flooding or erosion problems.
2. Develop a detailed hydrologic/hydraulic model of the watershed. Early planning should include flow and rainfall measuring equipment installed in the field, for calibration.
3. Review alternative management strategies that are feasible to implement based on watershed characteristics (i.e. soils, karst, and existing development). It may be prudent to perform this step concurrent with Step 2 to assure model development (specifically related to rainfall-runoff component) is consistent with methodology for selected alternative strategies.
4. Select a baseline goal for the point(s) of interest. This could be based at a point in time, or capacity availability. For a point in time, modify the watershed model to

represent the baseline condition in the watershed. This approach could utilize historic data (i.e. aerial photography) to modify land cover conditions and to modify the conveyance system infrastructure within the watershed.

5. Route the baseline condition model for a wide range of storm events.
6. In addition to existing facilities, provide “dummy” ponds and applicable areas for areas that are subject to stormwater management regulations with future development. Ideally, a pond would be provided for each potential development *site* and its corresponding basin. Placement of dummy applicable areas should consider areas that may practically be developed in the next 20-30 years. This will give a more accurate depiction of the strategies downstream performance potential by considering the reality of what portions of the watershed may, or may not, actually be subjected to stormwater management.
7. Create a copy of the base model for each management strategy. Design each pond within each model based on the specified design criteria for the corresponding strategy. Assure that design only meets the minimum requirements of the strategy (i.e. allow maximum release of the 25-year design storm if no reduction requirement is stated for this storm). This approach will most accurately represent the strategies implementation in practice.
8. Route a wide range of storms events with each model. It was found during the literature review of alternative strategies that some strategies were developed using SCS hydrology. The selection of rainfall should consider the implementation of the management strategies. However, it is recommended that measured rainfall should also be routed, especially to evaluate performance for the smaller, more frequent storm events (< 1-year).
9. Compare results at the point, or points, of interest. Any regional facilities within the watershed should be evaluated to determine any effects the alternative strategies may have on these facilities (e.g. flooding?).
10. Evaluate results based on a strategy's ability to meet the baseline goals established in Step 4. In addition to peak reduction at the point of interest, also consider the duration and volume that occurs near bank-full conditions. Volume

and cost requirements for implementation at the site scale should also be considered in context to the benefit gained at the point of interest.

11. Select the optimal management strategy. Based on the results analysis, modify the selected strategy, if needed, by adding requirements that may help meet baseline goals not being met with the strategy as is.
12. Create a new model and modify each control structure to represent the modified strategy. Verify if the baseline goal targeted by the modification is met. Also, assure that other goals that were previously met are not compromised.
13. If the modified strategy provides the desired results, perform a required volume and cost analysis to assure the cost/benefit analysis with the modification is still adequate.

6 Discussion and Conclusions

Chapter 6 provides a review of the objective presented in Chapter 1. Based on the analysis presented and specifically the evaluation of results, concluding thoughts and suggestions for future research are also provided.

6.1 Review of Objectives

The overall goal of this research is to investigate downstream runoff peak reductions resulting from application of alternative stormwater management strategies implemented throughout a branch of the Stroubles Creek watershed in Blacksburg, Virginia. The research achieved this goal and the specific objectives presented in Chapter 1. Specific tasks to accomplish the objective are summarized below.

- *Utilize a hydrologic and hydraulic model representative of the subject watershed with detail and scale adequate to represents the implementation of stormwater management within the watershed.*

  A SewerGEMS model was developed with detailed data that included the storm sewer system and management facilities within the watershed. The model included storm sewer infrastructure data collected in the field and supplemented with record drawings. All relevant information related to stormwater management facilities was also included. Runoff coefficient data was generated using land cover and soils data. Time of concentration calculations were performed using available mapping. Sub-basins were delineated to the scale of applicable areas. Several dummy facilities were added to provide more managed area, and therefore more defined results downstream. Finally, the model was calibrated using available measured data.

- *Review alternative stormwater management strategies and develop procedures to represent these strategies in the model within applicable areas.*
  Chapter 2 provides a review of the alternative stormwater management practices found in the literature. Chapter 3 provides the methodology for representing these strategies within the model.

- *Evaluate characteristics of runoff hydrographs resulting from each applicable area with each alternative management strategy. Routing will be subject to a range of design storms.*
  Chapter 4 provides an evaluation of characteristics observed with the implementation of each management strategy at the site scale.

- *Analyze the resulting hydrographs at the watershed outfall generated from implementation of each alternative management strategy for all applicable areas within the watershed.*
  Section 4.2 begins the discussion of the analysis of results at the watershed outfall. This section provides observations related to the results seen at the watershed outfall for implementation of each alternative strategy within the watershed. Representation of each strategy at a watershed scale was a result of all applicable areas being subjected to the design requirements for each strategy. The analysis resulted in a selection of the optimal strategy for the watershed and included a modification to the strategy based on observations from the analysis.

6.2 Final Thoughts

The results from this research provide some insight into what effect the implementation of alternative management strategies can have downstream, specifically in an urban watershed with only a portion of the total area subjected to stormwater management. At a watershed scale, it was found that none of the management strategies met the baseline goals for the full range of design storms. The goals appear obtainable with the larger, less frequent events, but appear more challenging with the smaller storms. As success with the larger storms is important to relieve downstream flooding concerns, the natural condition of streams is compromised with the smaller, more frequent storms. Although in the urbanized watershed, the natural stream condition may already be gone, natural conditions possibly still exist downstream and impact can be minimized if these more frequent storms can replicate the predevelopment condition during bank-full conditions.

Of course, frequency of runoff will increase with urbanization despite stormwater management. Since the frequency of runoff events can only be maintained to the predevelopment condition when runoff volumes are reduced to predevelopment conditions, only LID offers the possibility to maintain frequency of runoff events. In spite of the findings for the subject watershed, LID offers the most robust management strategy where it can be successfully applied. Bosley (2008) shows that when compared to a forested condition over a 77-month simulation period, traditional management strategies result in 10.6 times the runoff volume, while LID produces 3.1 times the runoff volume. This performance by a representation of an LID strategy at the site scale clearly illustrates the benefit of LID. However, Bosley also acknowledges that it is unclear if an LID strategy can successfully manage flood peaks resulting from larger, less frequent storm events.

Considering comparison of implementation of all the strategies presented, the range in peak runoff rates at the watershed outfall were relatively small, with the average variation to the predevelopment peak averaging 4% for the smaller storms (1- and 2-year) and 2.3% for the larger storms (10- and 25-year). This indicates that the portions of the watershed not subjected to stormwater management (75% of total area) provide most of the contribution to the peak at the watershed outfall. However, the difference in range between implementation of the varied strategies range as high as 21% for the 1-year storm (difference between Stratified and LID) or 11% (Stratified and Traditional with Water Quality) when LID is excluded from comparison. The difference in peak runoff between the Stratified and Traditional with Water Quality was also significant for the 2-year storm at approximately 16%. The range in results for larger storms was less dramatic. These results can be explained by the fact that storage within the watershed is exceeded with the pre- *and* post-development condition with the larger storms, with the available storage being relatively small compared to runoff volumes. However, this would not be the case with smaller storms since the available storage would account for a higher percentage of the total runoff volume, resulting in more variation in runoff between pre- and post conditions. Even though the range of peak runoff results is relatively small with implementation of the management strategies, recall that there is a 25% increase in peak runoff rates at the watershed outfall when no management is in place.

It appears that detention cannot provide the controls necessary to meet the baseline goals for the 1-year storm, no matter how stringent the requirements become (e.g. 72-hour drawdown on the EURV + 10%). The 1-year watershed baseline goal was only met when *volumes* were reduced (retained on-site) that approximate the difference between the pre- and post development runoff for the 1-year design storm. This indicates that additional runoff generated from the loss in available storage, due to the developed condition, is significant enough to contribute to the downstream peak no matter how small the rate of release from the site. Interestingly this occurs with only 25% of the watershed subjected to management and therefore only the land cover from this 25% having different land cover conditions compared to the baseline condition. This suggests that after only a portion of the watershed is developed, the baseline target for smaller storm events become unobtainable. It also suggests that any retention that can be provided on-site should be encouraged. This is because although it was found that the full difference in the pre- and post volumes needed to be retained to meet the 1-year watershed baseline, the retention of a percentage of this volume may help meet watershed goal for storms smaller than the 1-year storm.

With the variability between watersheds, alternative management strategies should be evaluated to determine which provides the optimal results at a point, or points of interest in a watershed. The subject watershed represents an urbanized area, only partially subjected to stormwater management. There are also two regional management facilities located in the watershed. It is suggested that the optimal strategy for implementation may vary based on the percentage of the total area subjected to stormwater management and the existence of regional facilities. An analysis similar to the one presented in this paper may prove worthwhile when considering some of the counter-intuitive results encountered. For example, even though the URR, FSD and VDP provide significant reduction below the predevelopment peak runoff rate at the site scale, the benefit is not transferred downstream. This illustrates that larger storage facilities that allowable smaller release rates, and consequently higher costs, do not provided added benefit downstream.

6.3 Future Research

The presented analysis provides insight for the subject watershed. However, the variability that exists across watersheds may result in the selection of a different tailored strategy. Future research should consider varying percentages of area subjected to stormwater management within a watershed. Watersheds at varying levels of urbanization should also be considered. Furthermore, recall that the Stratified watershed analysis for development of release rates resulted in all of those applicable areas in which the discharge eventually was conveyed through a regional facility being allowed a release rate of 100% of the predevelopment release rate. In contrast, all those applicable areas that were not subject to a downstream release rate were only allowed to release a percentage (60 – 90%) of the predevelopment release rate. Since these release rates were developed based on the applicable areas contribution to the peak runoff rate at the watershed outfall, this indicates that regional facilities do play a role in regards to having an effect on the downstream peak. Future research should provide a closer examination of the effect of the facilities within the watershed. The examination should also consider if management strategies can be modified or varied throughout a watershed based on the presence of downstream regional facilities.

7 References

Behera, P. K., Papa, F., and Adams, B. J. (1999). "Optimization of Regional Storm-Water Management Systems." *Journal of Water Resources Planning and Management*, 125(2), 107-114.

Bosley, Eugene K. (2008). *Hydrologic Evaluation of Low Impact Development Using a Continuous, Spatially Distributed Model*. MS Thesis, Dept. of Civil and Environmental Engineering, Virginia Polytechnic Institute and State University.

Brown, W., and T. Schueler. 1997. T*he Economics of Stormwater BMPs in the Mid-Atlantic Region.* Prepared for Chesapeake Research Consortium. Edgewater, MD. Center for Watershed protection. Elliot City, MD.

Carter, S., Zhen, J., Parker, A., and Cutter, B. "An Innovative Stormwater Best Management Practice Decision Support System for Quantifying and Optimizing Load Reductions and Costs in Los Angeles." *World Environmental and Water Resources Congress 2008*, Ahupua'a.

Emerson, C. H., Traver, R. G., and Welty, C. (2004). "Changing from Peak Flow to Volume Requirements, a Watershed-Wide Perspective."

Emerson, C. H., Welty, C., and Traver, R. G. (2005). "Watershed-Scale Evaluation of a System of Storm Water Detention Basins." *Journal of Hydrologic Engineering*, 10(3), 237-242.

EPA, 1999. *Preliminary Data Summary of Urban Storm Water Best Management Practices.* EPA 821 -R-99-012.

Fennessey, L. A. J., Hamlett, J. M., Aron, G., and LaSota, D. (2001). "Changes in Runoff due to Stormwater Management Pond Regulations." *Journal of Hydrologic Engineering*, 6(4), 317-327.

Goff, K., and Gentry, R. (2006). "The Influence of Watershed and Development Characteristics on the Cumulative Impacts of Stormwater Detention Ponds." *Water Resources Management*, 20(6), 829-860.

Henrico County Department of Public Works Division. (2001) "Henrico County Environmental Program Manual."

Kaini, P., Artita, K., and Nicklow, J. W. (2007) "Evaluating Optimal Detention Pond Locations at a Watershed Scale." 170.

Lai, F.-h., Shoemaker, L., and Riverson, J. "Framework Design for BMP Placement in Urban Watersheds." *World Water Congress 2005*, Anchorage, Alaska, USA, 185-185.

Lehigh Valley Planning Commission. (2000). "Little Lehigh Creek Watershed - Act 167 - Stormwater Management Plan Update." L. V. P. Commission, ed.

Malcom, H. R. (1980). "A study of detention in urban stormwater management ", Water Resources Research Institute of the University of North Carolina.

McCaffery, K. A. "Comprehensively Establishing Predevelopment Hydrologic Conditions : A Multiple-Scale Approach for Responding to Trends in Stormwater Management Regulations." *World Environmental and Water Resources Congress 2007*, Tampa, Florida, USA, 576-576.

McCuen, R. H. (1979). "Downstream Effects of Stormwater Management Basins." *Journal of the Hydraulics Division, ASCE*, 105(HY11).

McCuen, R. H., and Moglen, G.E. (1988). "Multicriterion stormwater management methods." *Water Resources Planning and Management*, 114(4), 414-431.

McEnroe, B. M., and Heatherman, W. J. "Simplified Detention Requirements with Watershed-Wide Benefits." *World Environmental and Water Resources Congress 2006*, Omaha, Nebraska, USA, 423-423.

Milwaukee Metropoplitan Sewerage District. (2005). "Surface Water and Stormwater Rules Technical Guidance." Milwaukee Metropoplitan Sewerage District.

Moore, C. I., Barrack Ii, W. A., and Meyers, M. "Innovative Modeling Techniques for Watershed Planning." *Managing Watersheds for Human and Natural Impacts*, Williamsburg, Virginia, USA, 131-131.

NAHB Research Center, I. (2003). "The Practice of Low Impact Development." U. S. D. o. H. a. U. Development, ed., U.S. Department of Housing and Urban Development.

Nehrke, S. M., and Roesner, L. A. (2004). "Effects of Design Practice for Flood Control and Best Management Practices on the Flow-Frequency Curve." *Journal of Water Resources Planning and Management*, 130(2), 131-139.

New Jersey Department of Environmental Protection Division of Watershed Management. (2004). "New Jersey Stormwater Best Management Practice Manual." M. S. C. O. f. Municipalities, ed.

Pazwash, H., and Boswell, S. T. "Design of Stormwater Management Systems: Suggested Improvements." *World Water Congress 2001*, Orlando, Florida, USA, 174-174.

Perez-Pedini, C., Limbrunner, J. F., and Vogel, R. M. (2005). "Optimal Location of Infiltration-Based Best Management Practices for Storm Water Management." *Journal of Water Resources Planning and Management*, 131(6), 441-448.

Prakash, A. "Impact of Urbanization in Watersheds on Stream Stability and Flooding." *Managing Watersheds for Human and Natural Impacts*, Williamsburg, Virginia, USA, 143-143.

Prince George's County. (1999). "Low-Impact Development Hydrologic Analysis." D. o. E. Resources, ed., Prince George's County, Maryland.

Riverson, J., Zhen, J., Shoemaker, L., and Lai, F.-h. "Design of a Decision Support System for Selection and Placement of BMPs in Urban Watersheds." *World Water Congress 2004*, Salt Lake City, Utah, USA, 40-40.

SCS. (1986). "Urban Hydrology for Small Watersheds." Soils Conservation Service, Washington, DC.

Shamsi, U. M. (1996). "Storm-Water Management Implementation through Modeling and GIS." *Journal of Water Resources Planning and Management*, 122(2), 114-127.

Southeastern Wisconsin Regional Planning Commission. (2007). "A Regional Water Quality Management Plan Update for the Greater Milwaukee Watersheds." Southeastern Wisconsin Regional Planning Commission.

State of Virginia. (1999). "Virginia Stormwater Management Handbook - Edition 1." Department of Conservation and Recreation, ed.

State of Virginia. (2001). "Virginia Stormwater Management Regulations." Department of Conservation and Recreation.

State of Virginia. (1992). "Virginia Erosion and Sediment Control Handbook." Department of Conservation and Recreation.

U.S. Department of Agriculture. (1986). "Urban Hydrology for Small Watersheds." Technical Release Number 55. National Technical Information Service.

Urbonas, B., and Wulliman, J. "Stormwater Runoff Control Using Full Spectrum Detention." *World Environmental and Water Resources Congress 2007*, Tampa, Florida, USA, 36-36.

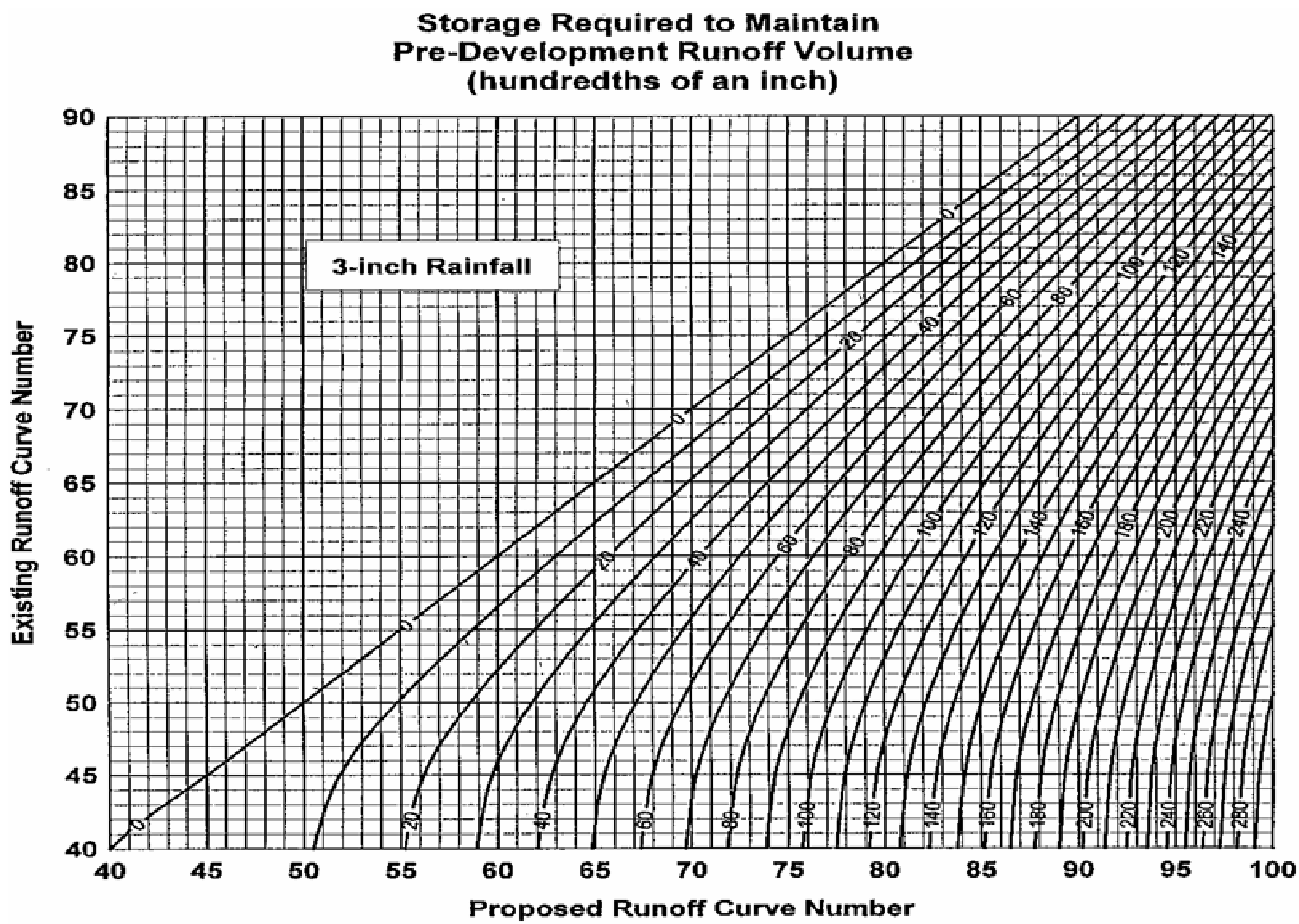
USEPA. (2002). "National management measures to control nonpoint source pollution from urban areas - draft." U. S. E. P. Agency, ed., Rep. EPA 842-B-02-003, U.S. Environmental Protection Agency Office of Wetlands, Oceans, and Watersheds, Washington, D.C., Washington, D.C.

Waukesha County. (2004). "Minutes - Waukesha County Stormwater Advisory Committee Meeting." Waukesha County Stormwater Advisory Committee, Waukesha County

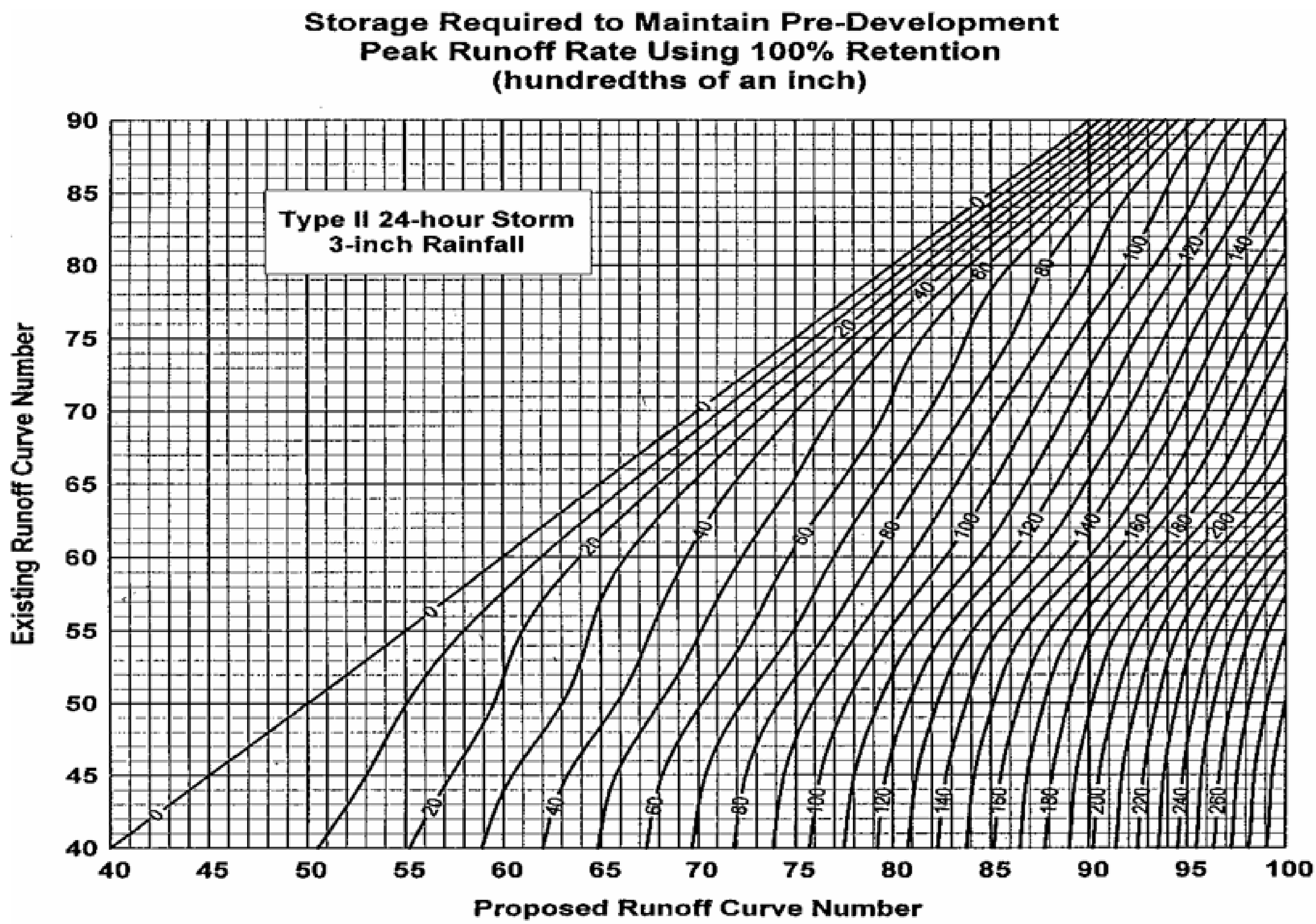
8 Appendix

Appendix A: LID Chart Series

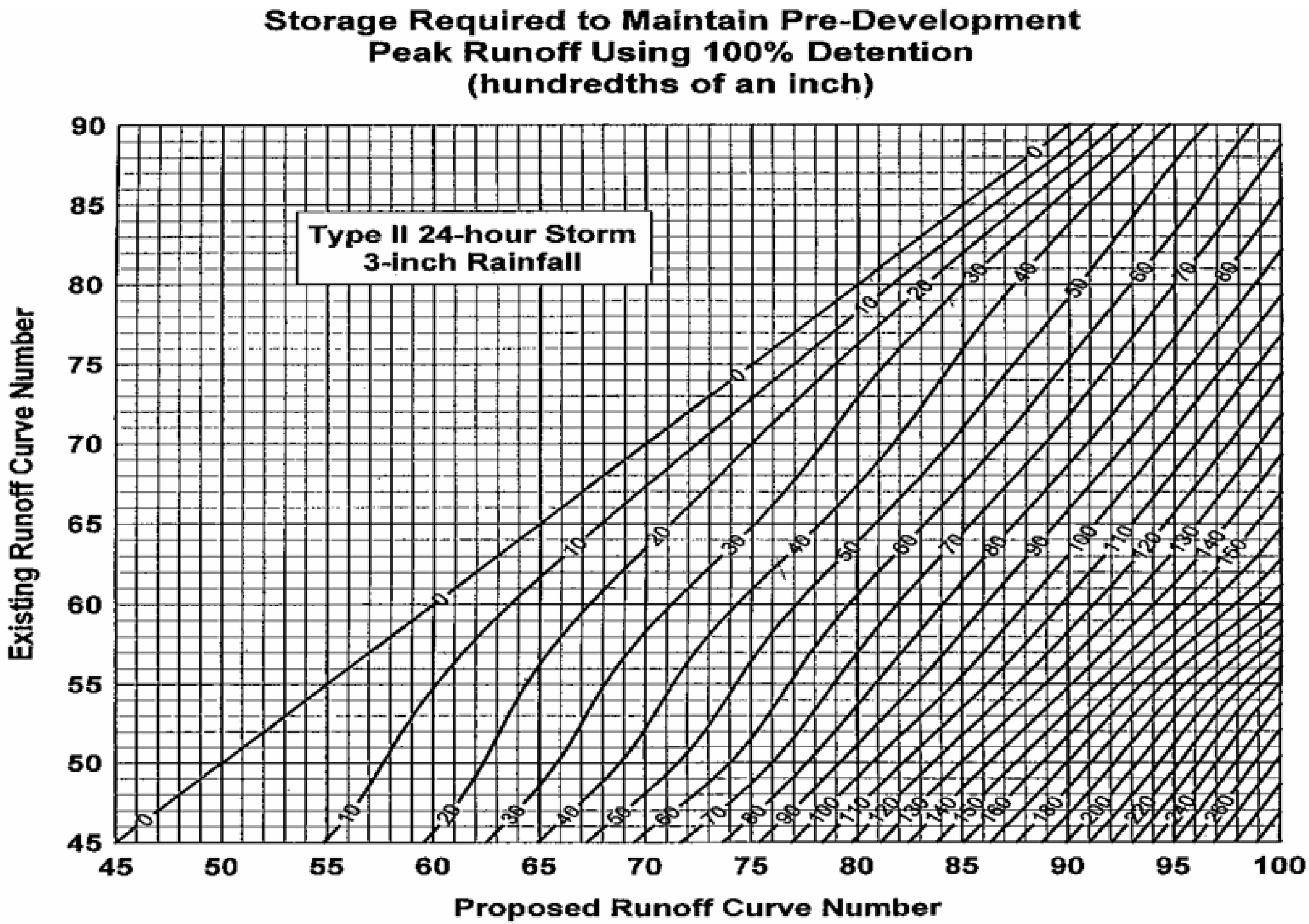
- **A-1: LID Chart Series: Storage Required to Maintain Pre-development Runoff Volume**
- **A-2: LID Chart Series: Storage Required to Maintain Pre-development Runoff Volume using 100% Retention**
- **A-3: LID Chart Series: Storage Required to Maintain Pre-development Runoff Volume using 100% Detention**



A-1. LID Chart Series: Storage Required to Maintain Pre-development Runoff Volume (PGC, 1999).



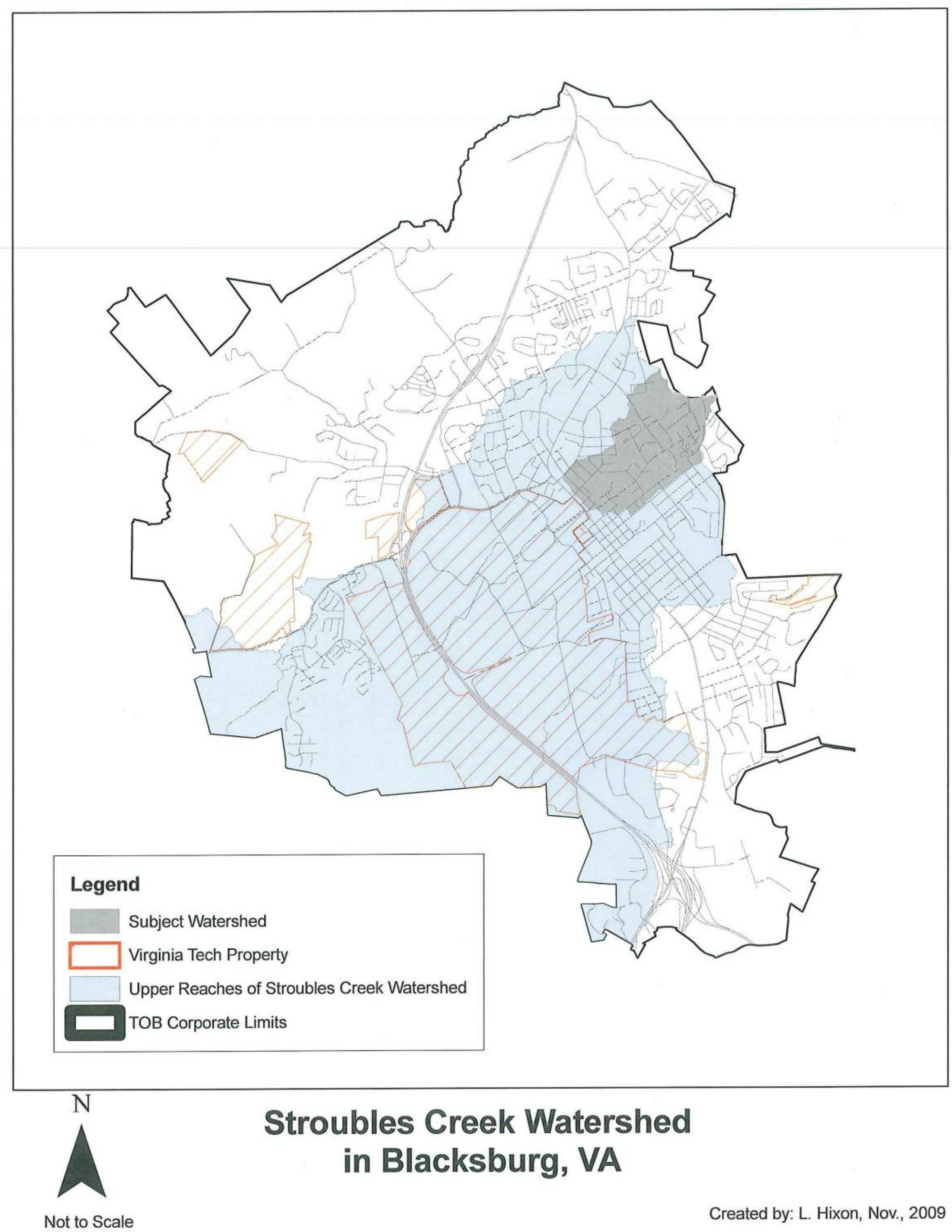
A-2. LID Chart Series: Storage Required to Maintain Pre-development Runoff Volume Using 100% retention (PGC, 1999).



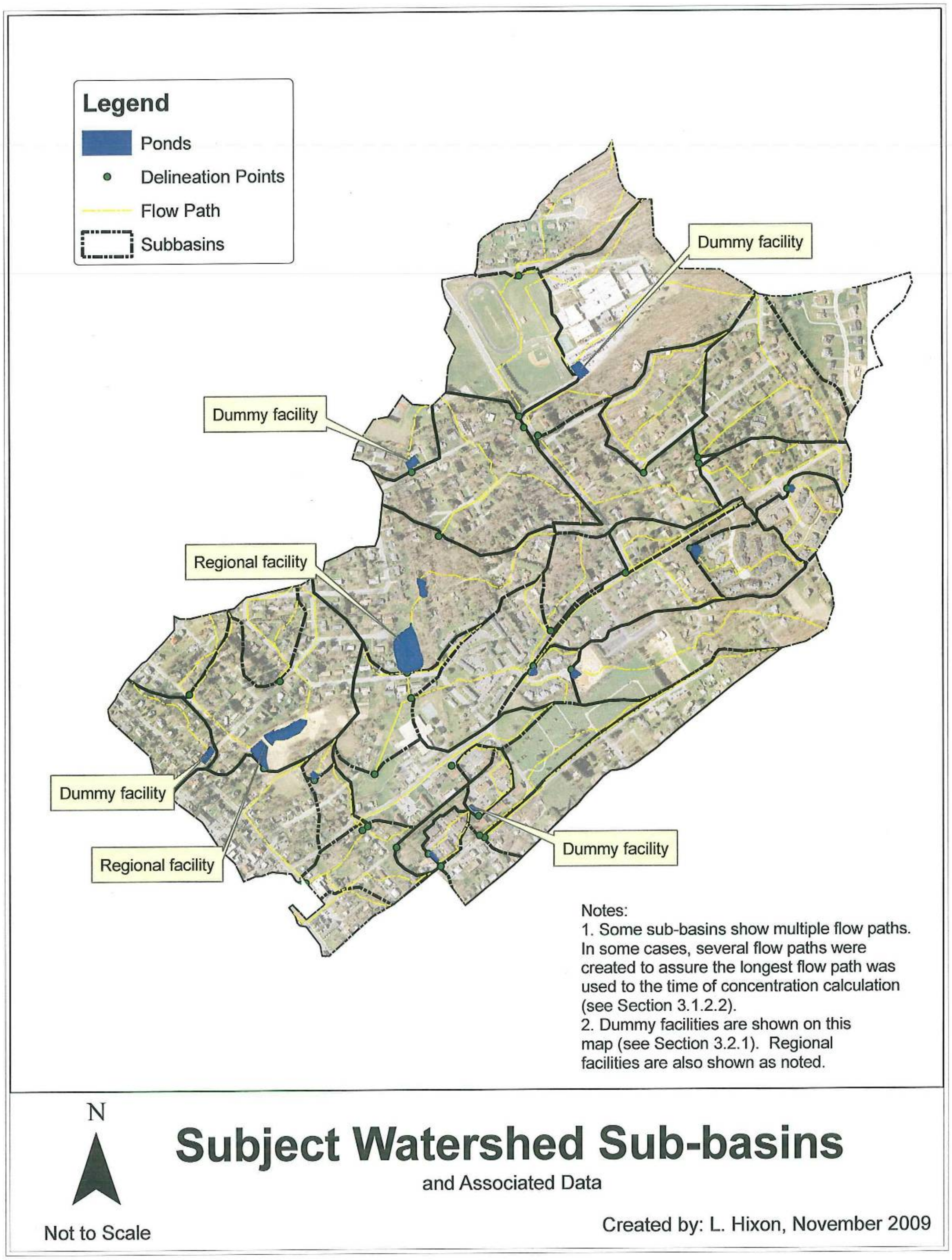
A-3. LID Chart Series: Storage Required to Maintain Pre-development Runoff Volume using 100% detention (PGC, 1999).

Appendix B: Maps of Subject Watershed

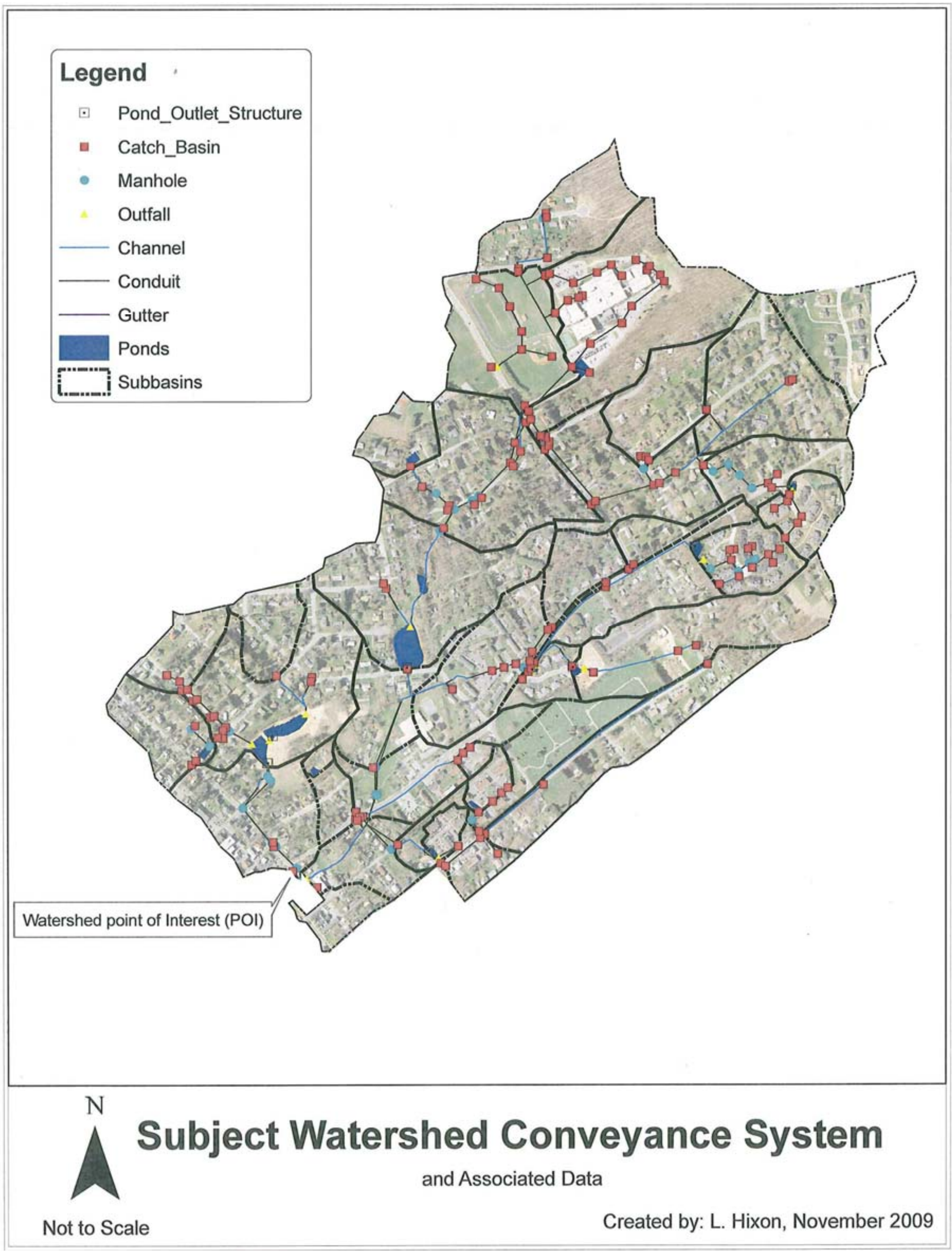
- **B-1: Subject Watershed**
- **B-2: Subject Watershed Sub-basins and ponds**
- **B-3: Subject Watershed Conveyance System**



B-1. Subject Watershed



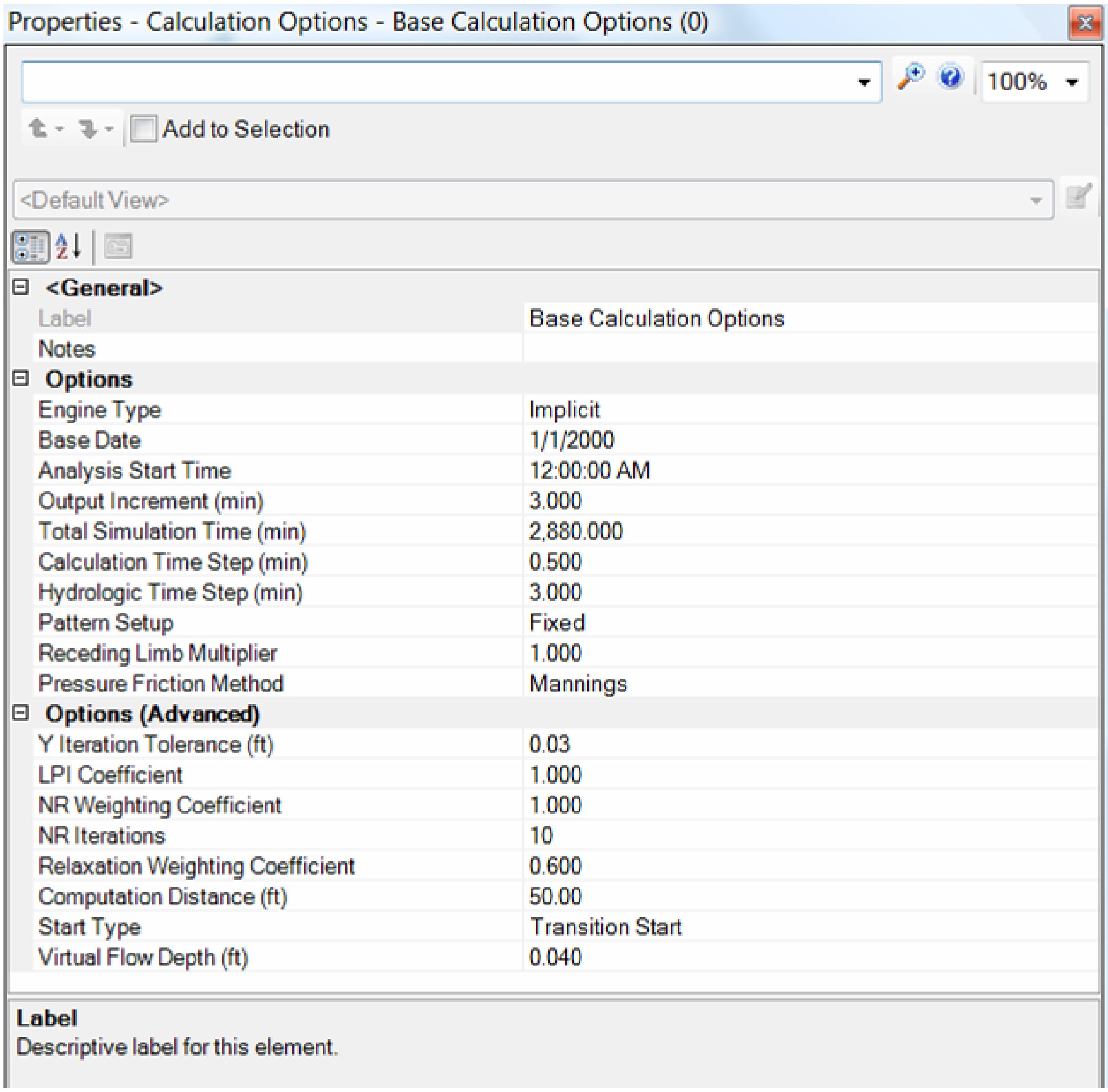
B-2. Subject Watershed Sub-basins and ponds



B-3. Subject Watershed Conveyance System

Appendix C: SewerGEMS Implicit engine option settings

- **C-1: Model Calculations Options**



C-1. Model Calculations Options

Appendix D: Curve number data for subject watershed (CGIT's CN Development Report

- **D-1: CGIT Curve Number Development Report**

(D-1. This cover page and next 12 pages)

Curve Number Development

in the Town of Blacksburg

Stormwater Management Research Report

Prepared by the Center for Geospatial Information Technology (CGIT) at Virginia Tech

July 2, 2008

Introduction

This report explains methods which may be used to develop land cover, and subsequently runoff curve number, datasets in GIS. These datasets are important components of many hydrologic models; curve number is the primary parameter governing the quantity of runoff generated in response to rainfall events, just as time of concentration is the primary parameter governing the rate of runoff from a watershed. This report is intended to be a guide to the development of such datasets using modern GIS resources. After reviewing SSURGO soils data usage, three different methods for acquiring land cover data are demonstrated. Finally, the curve numbers that result from intersecting these different land cover datasets with the soils data are also compared.

To develop a stormwater model, estimates of runoff volumes expected from given rainfall events must be developed based on the ground conditions in each watershed (or other unit of analysis). Many methods to calculate runoff exist, with varying levels of detail. One of the more common methods is the SCS/NRCS Curve Number method, detailed in the NRCS Technical Release 55 (TR-55)[1]. In this method, land cover conditions and generalized hydrologic soil groups are combined to determine a "curve number," which is a key parameter in the TR-55 runoff equation. The curve number does not have units, but typically ranges from 30 – 98, with higher numbers producing greater volumes of runoff. The TR-55 manual includes tables of land cover conditions, hydrologic soil groups, and the resultant curve numbers. To use these tables to estimate curve number in a GIS environment, polygon (or raster) datasets of hydrologic soil group and land cover condition should be developed independently and then intersected to yield curve numbers.

Hydrologic soil group information can be acquired from SSURGO (Soil Survey Geographic) databases developed by the USDA NRCS. The SSURGO data is the most detailed soil survey data available in most areas. Without SSURGO data, it would be necessary to conduct an independent survey of soil infiltration rates across the study area.

Land cover condition data can be developed more cheaply than soils data, and may be developed using a variety of methods. At a nationwide level, the Multi-Resolution Land Characteristics Consortium (MRLC)[2] produces the National Land Cover Database (NLCD) line of products, derived from satellite imagery. The primary NLCD product is a land cover classification raster with a 30 meter cell size; additional datasets of imperviousness and forest canopy have also been developed. At a local level, land cover data may be developed from interpretation of aerial imagery, compilation of existing GIS data, or some combination thereof. The use of infrared aerial imagery could offer some benefit for distinguishing between vegetated and non-vegetated areas. Of course, the feasibility of developing land cover datasets at a local level depends on the quality of GIS data available for the area, as well as the resources available for development and maintenance of the dataset.

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[1] USDA NRCS (US Department of Agriculture Natural Resources Conservation Service). 1986. Technical Release 55: Urban Hydrology for Small Watersheds.
[2] The MRLC is a collection of federal agencies; the consortium website is http://www.mrlc.gov

Soils Data

SSURGO soils databases, typically published on a countywide basis, contain a variety of attributes relating to engineering and agricultural activities. The SSURGO data is often the most detailed soils information available for any given study area, short of undertaking a costly new soil survey. SSURGO's tabular data can be related to the SSURGO spatial map unit polygons, although often the relationship between map unit polygons and records in a SSURGO table is typically not 1:1. Hydrologic Soil Group (HSG) data is reported in the soil component table, but many soil components may be present in a map unit polygon. Therefore, to create a GIS dataset of HSG polygons, data aggregation within the SSURGO database is necessary.

A number of new options now exist for performing SSURGO data aggregation and mapping. The SSURGO Soil Data Viewer (currently version 5.2), an extension for ESRI's ArcMap software, can perform aggregation and mapping of soil attributes using a few common methods. A popular aggregation method is "Dominant Condition," in which the attribute with the highest representation from among the various soil components present in a map unit is selected. For example, if 75% of the soil components present in a map unit have an HSG of C, then the entire map unit will be assigned an HSG of C. In addition to the aggregation tools available through the Soil Data Viewer for ArcMap, newer SSURGO database exports also include a table named "muaggatt," which contains some common soil attributes aggregated to the map unit level. This table includes aggregated HSG values which can be easily joined to the map unit polygons for use in GIS.

In the Montgomery County, Virginia SSURGO database, a few map unit polygons are composed primarily of "Udorthents and Urban land." These areas include disturbed soils, fill dirt, and other mixes of soil that are not naturally occurring. Geographically, these map units are mostly located in developed areas, as would be expected. The SSURGO database does not attempt to estimate a hydrologic soil group for these map units. For the purpose of determining curve numbers, these areas are commonly assumed to be HSG D, as this is the most conservative approach from an engineering standpoint. To account for any variation within these map units, it would be necessary to conduct a detailed field survey.

Generalized Land Cover

The "generalized" land cover dataset was developed using approximately the same level of detail as is commonly used for performing TR-55 curve number estimates for engineering designs. This method classifies homogeneous areas of suburban development into polygons using classifications drawn largely from TR-55 table 2-2a (e.g. Residential 1/3 acre lots, Commercial and business, etc.). Since the method is performed at the neighborhood scale or broader, it can be applied to a large area fairly quickly.

To carry out this method for the Town of Blacksburg, the 2005 aerial imagery collected during the countywide LiDAR project was interpreted by student workers, who digitized polygons and assigned attributes. The digitization was performed in an ArcGIS Personal Geodatabase, with a domain table to standardize polygon attribution, and a topology established to ensure polygon boundary snapping.

Detailed Land Cover

The "detailed" land cover dataset is an attempt to represent specific land cover conditions at the spatial resolution of individual sidewalks and driveways. This method bypasses the generalized residential lot size categories included in TR-55 table 2-2a, and should provide a more accurate delineation of true imperviousness. Due to the level of detail, this method was initially applied only to a selected subwatershed within the Town.

The detailed land cover dataset was developed by supplementing the existing road edges and buildings GIS datasets. Based on the 2005 aerial imagery, student workers added edge lines to delineate individual features. First, they focused on "hardscape" elements such as sidewalks, driveways, buildings, and other constructed features. After this was complete, they divided the remaining "softscape" areas between the various vegetated cover types. This two-pass approach prioritizes the hardscape features, and ignores the effects of tree canopy cover over hardscape features. Thus, this dataset assumes that tree canopy interception above impervious surfaces is not important.

The digitization work was performed in a slightly different manner than the generalized dataset. Rather than digitizing polygons, the student workers digitized edges that snapped to form closed figures. Area points were added to the interiors of these closed figures, containing the land cover attributes to be applied to that area. Finally, the edge lines were converted to polygons using the ArcToolbox "Feature to Polygon" tool, with the area points providing the polygon attributes. This method is often preferable when digitizing a large number of features, because direct polygon editing can become inefficient and troublesome, due to the necessity for shared boundaries.

NLCD Land Cover

The National Land Cover Database, produced by the Multi-Resolution Land Characteristics Consortium (MRLC), is a raster dataset depicting land cover conditions for the nation. The dataset is based on interpretation of Landsat satellite imagery. The 30 meter cell size of the NLCD data is much coarser than the detailed method described above, but is comparable to the generalized method. No additional processing, other than assignment of NLCD classes to TR-55 classes, is required prior to using this data in a raster analysis environment. Table 1 and Figure 1 illustrate the land cover reclassification.

Table 1. Conversion between NLCD and TR-55 land cover classes.

NLCD 2001 Class	TR-55 Class
Open Water	Impervious-paved parking
Developed, Open Space	Open space
Developed, Low Intensity	Open space
Developed, Medium Intensity	Open space
Developed, High Intensity	Urban Districts-Industrial
Barren Land (Rock/Sand/Clay)	Newly Graded Areas (pervious areas, no vegetation)
Deciduous Forest	Woods
Evergreen Forest	Woods
Mixed Forest	Woods
Shrub/Scrub	Brush
Grassland/Herbaceous	Pasture,grassland,range
Pasture/Hay	Pasture,grassland,range
Cultivated Crops	Row Crops,Straight Row
Woody Wetlands	Impervious-Street/Roads-Dirt
Emergent Herbaceous Wetlands	Impervious-Street/Roads-Dirt

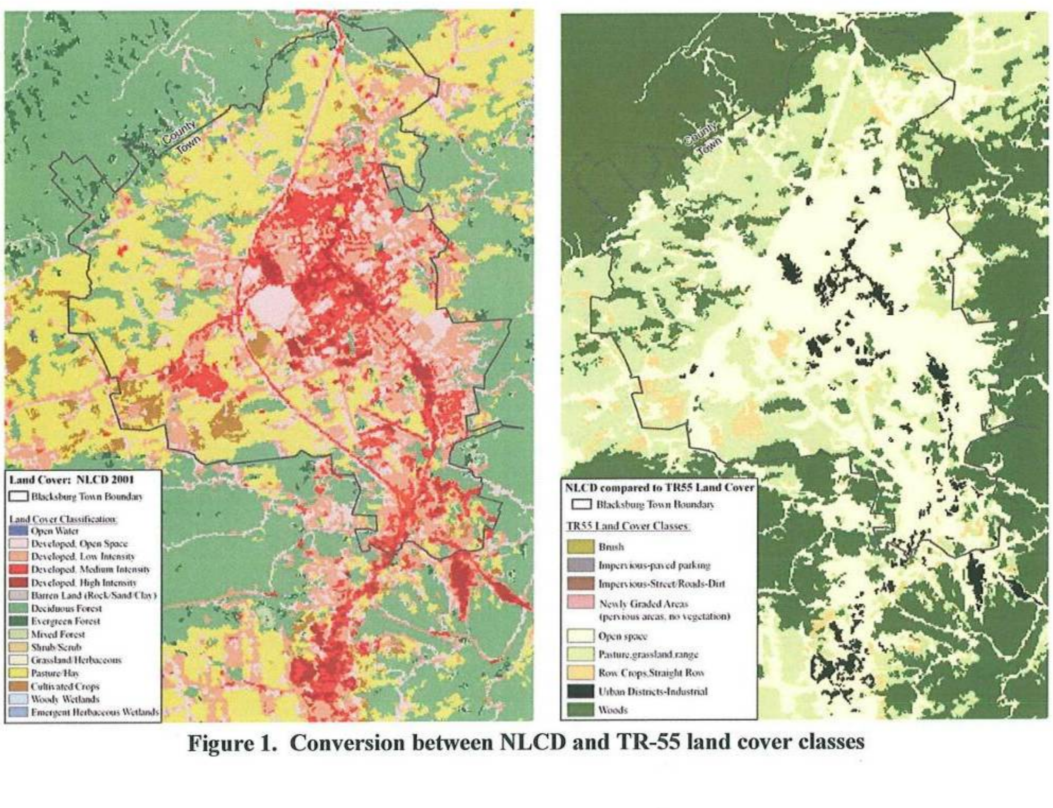


Figure 1. Conversion between NLCD and TR-55 land cover classes

Methods for intersecting Land Cover and Soils to generate Curve Number

The method used to calculate specific curve numbers for the vector datasets (generalized and detailed) is a series of table joins and vector geoprocessing operations. First, the land cover polygons are intersected with the soil HSG polygons. Second, a table is developed to link unique combinations of land cover and soil HSG to the output curve numbers. This table is joined to the result of the polygon intersection.

The method used to calculate specific curve numbers for the raster dataset (NLCD) is a series of table joins, reclassifications, and raster calculator operations. First, the soil HSG data must be rasterized using the same cell size and alignment as the NLCD data. Second, raster calculator is used to create a raster with values representing a concatenation of the land cover and soil codes. This result can be reclassified, using a table that relates these codes to curve numbers.

Comparison of Results

Comparisons between the generalized, detailed, and NLCD methods were performed on an area-weighted average basis for selected subwatersheds in the Town. The results of the comparison were interesting; despite the different methods and level of detail used in development of the various land cover datasets, the curve numbers tended to average out to approximately the same value within a given subwatershed. While the differences between datasets are significant at the scale of one parcel, when viewed at the scale of a neighborhood or subwatershed, the differences tend to balance and yield the same area weighted average.

Table 2 and Figure 2 show the results of a comparison of the three methods within the area selected for the detailed method. Table 3 and Figure 3 show the results of a comparison between the generalized method and the NLCD method; these are the only two methods which have been completed for the entire Town.

When comparing the NLCD method to one of the digitized methods, it is important to remember the source dates. The NLCD is a depiction of land cover conditions in 2001, while the digitization methods (generalized and detailed) were based on 2005 aerial imagery. A number of developments occurred in the Town between 2001 and 2005, which makes direct comparison unreasonable. Similarly, development has continued since 2005, which means that even the digitization methods are not completely reflective of current conditions. Maintaining a land cover dataset that is up-to-date all the time would be difficult, and probably unnecessary.

Table 2. Comparison of curve number methods for subwatersheds 15, 16, and 75.

	Generalized Method			Detailed Method			NLCD 2001		
Subshed	*Mean**	*Min*	*Max*	*Mean**	*Min*	*Max*	*Mean**	*Min*	*Max*
15	79.52	55	98	80.29	48	98	77.08	61	89
16	83.42	74	94	82.25	65	98	79.68	70	89
75	82.79	55	98	82.88	48	98	80.10	61	93

* Area Weighted Curve Number

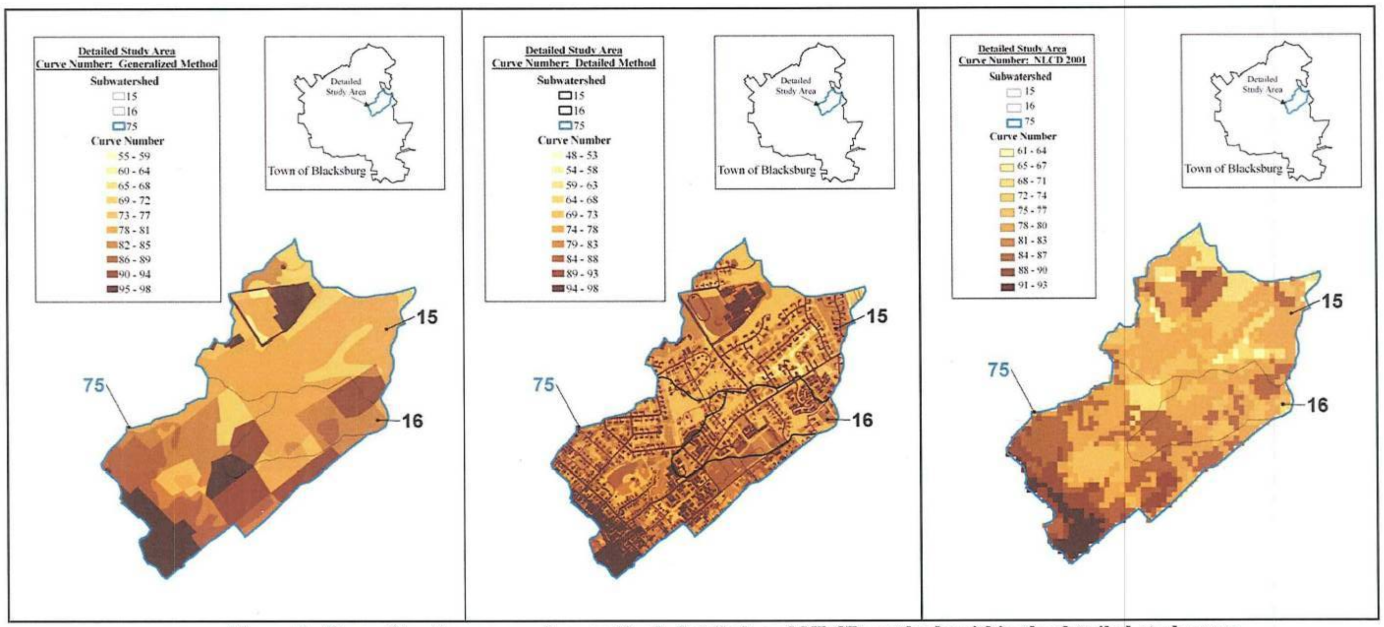


Figure 2. Curve Number maps of generalized, detailed, and NLCD methods within the detailed study area.

Table 3. Comparison of curve number methods outside detailed study area.

	Generalized Method			NLCD 2001		
Subshed	*Mean**	*Min*	*Max*	*Mean**	*Min*	*Max*
7	84.79	74	98	82.58	74	91
68	84.60	80	85	85.04	79	93
131	76.48	55	98	73.60	55	89
136	82.94	72	90	79.63	55	86
141	82.98	70	98	80.27	70	93
147	67.52	55	80	77.23	55	86

*Area Weighted Curve Number

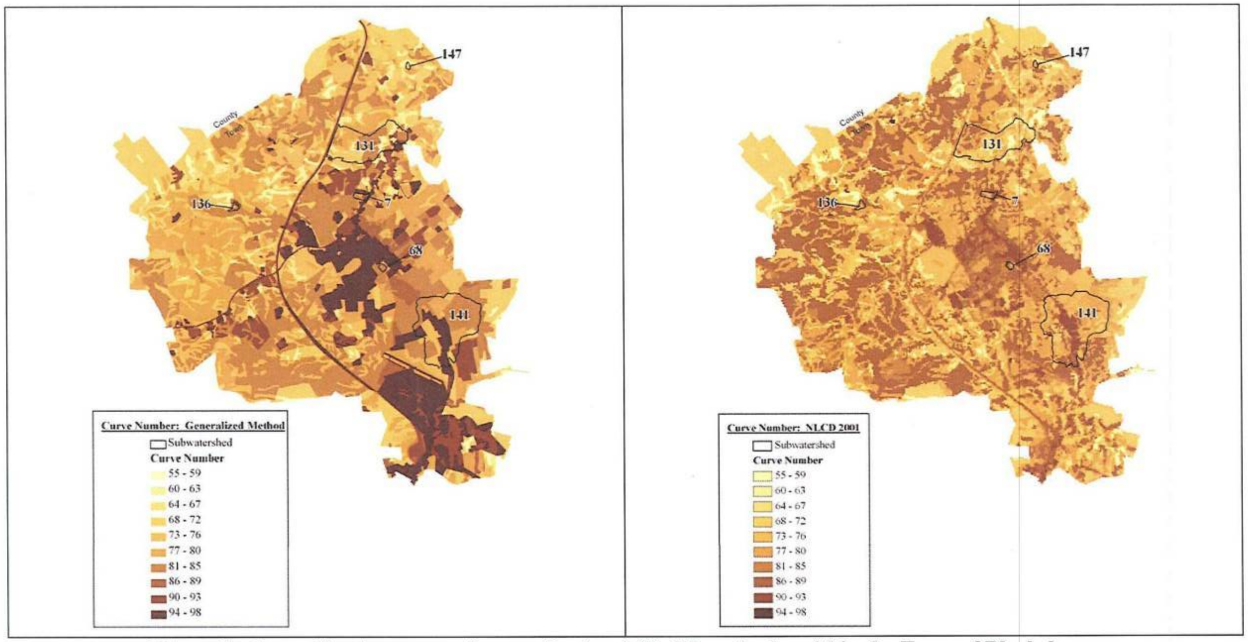


Figure 3. Curve Number maps of generalized and NLCD methods, within the Town of Blacksburg.

Datasets Conveyed with this Report

The datasets documented in this report which are being delivered include:

Generalized Method:
Gen_CurveNumber_town_u83z17.shp (vector data, includes Curve_Numb attribute field)
Curvenumbertable_v2.dbf (table used to relate land cover and HSG to curve number)

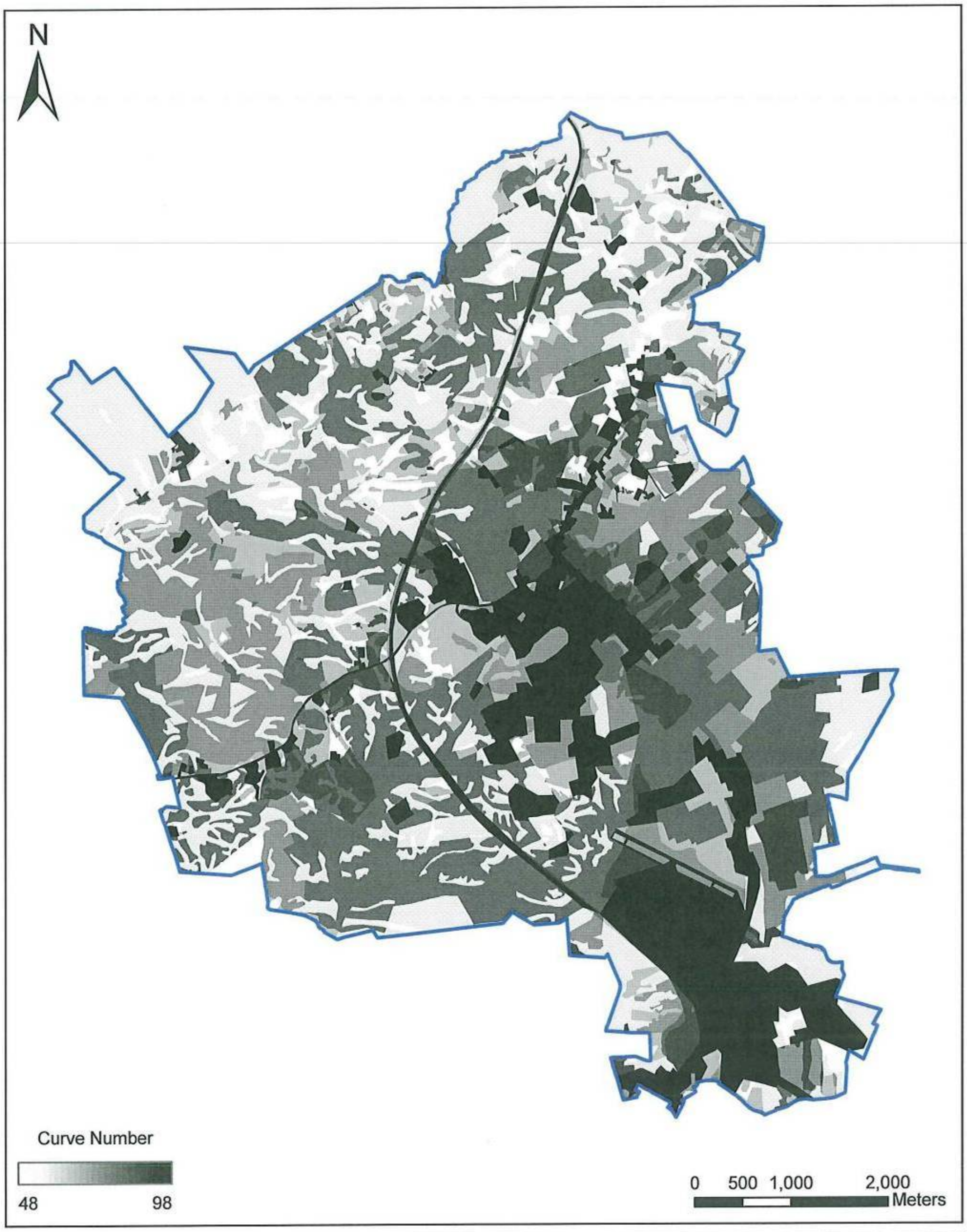
Detailed Method:
CurveNumber_Detailed_u83z17.shp (vector data, includes Curve_Numb attribute field)
Detailed_Curve_Number_Table.dbf (table used to relate land cover and HSG to curve number)

NLCD Method:
Cnnlcdbburg (raster data, value field of the raster is the Curve Number)
NLCD_classes_jointable-CurveNumber.xls (relates NLCD classes and HSG to curve number)
NLCD_classes_jointable-CurveNumber.dbf (simplified version of above table, used in GIS)

Soils Data:
HydGrp_SoilDataViewer-DomCondition_u83z17.shp (vector data, contains original Hydrologic Group field from SSURGO's muaggatt table and a modified HSG field in which blanks are replaced with "D")

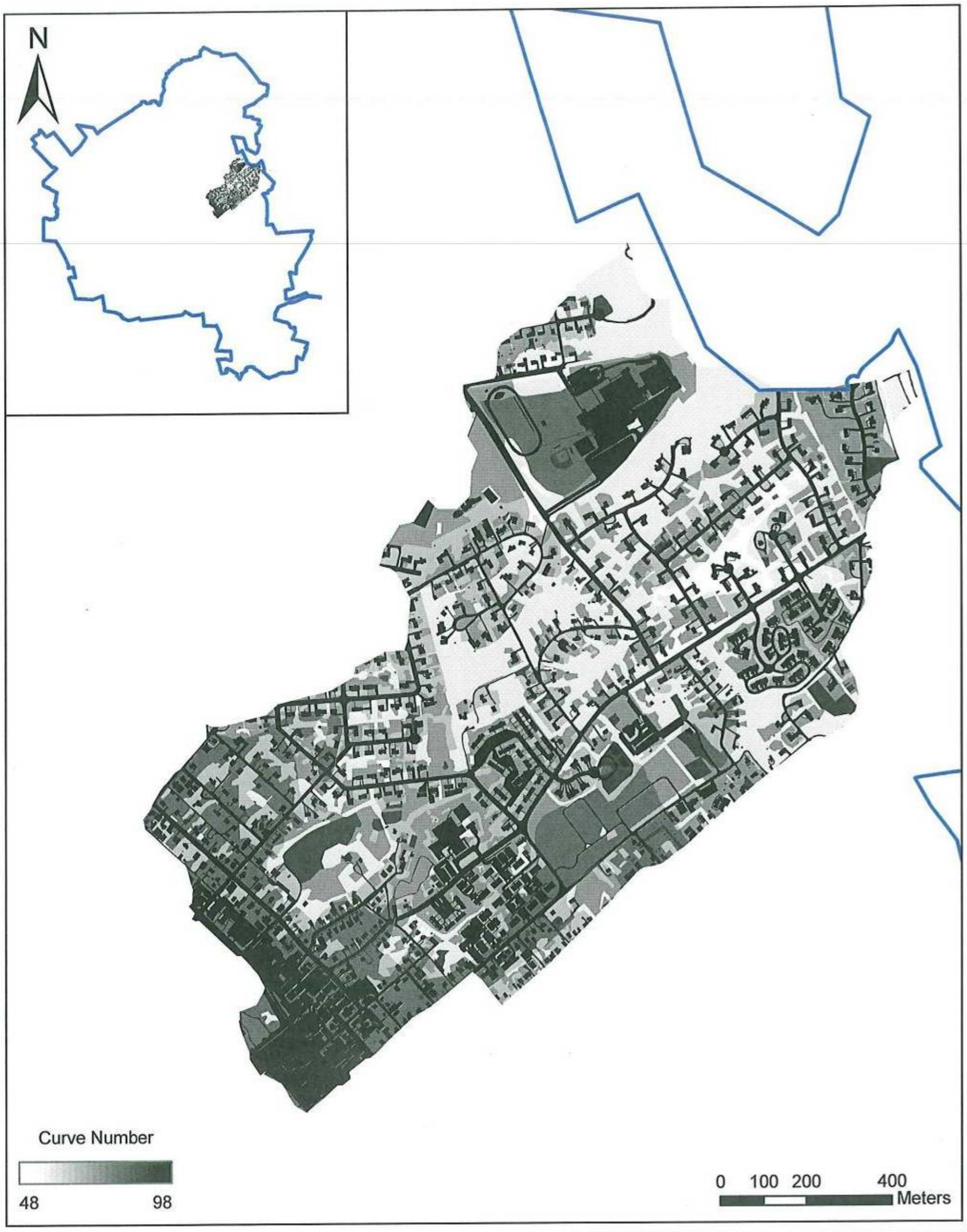
Maps

A general overview map of each method is presented on the following pages.



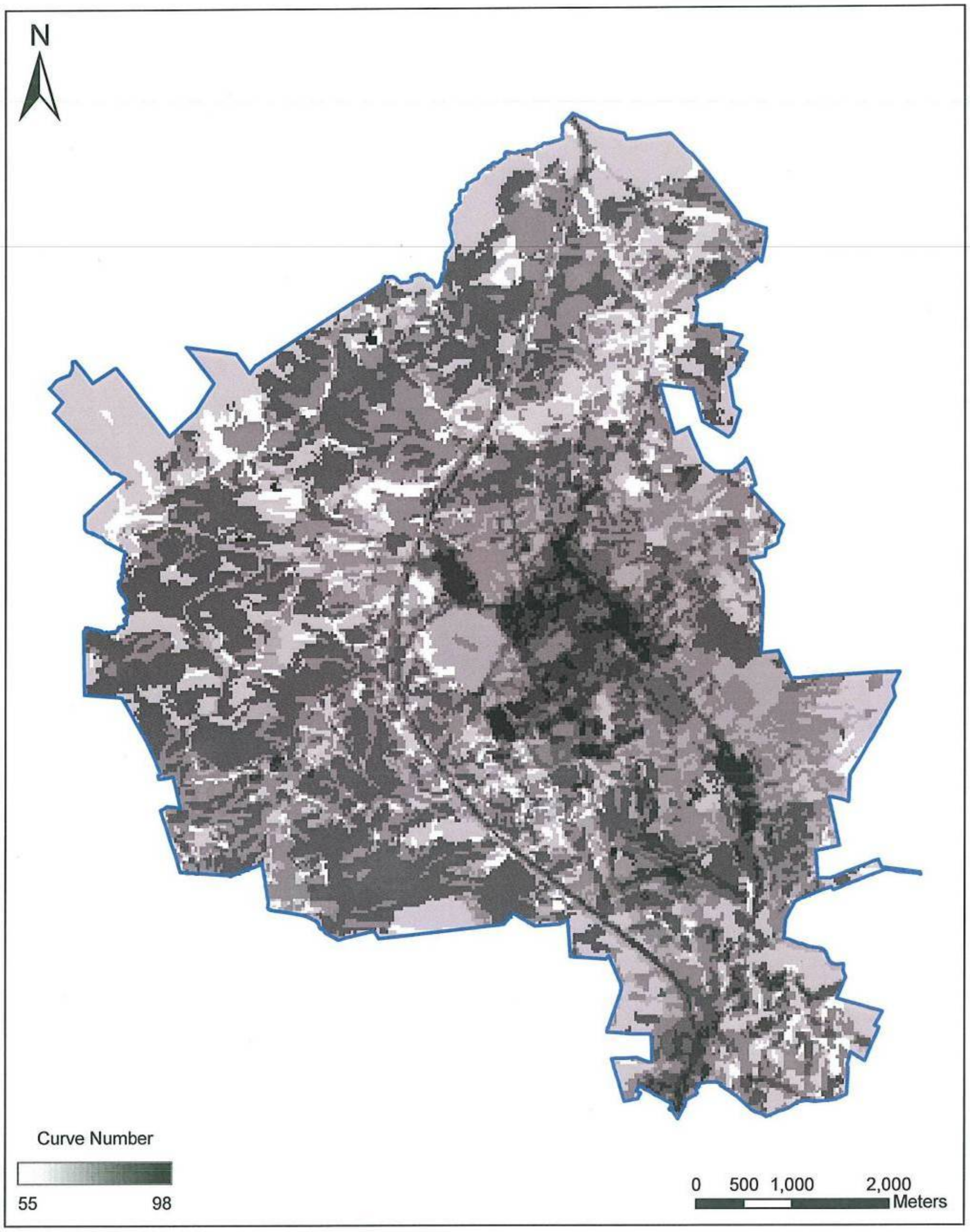
Town of Blacksburg Curve Number: Generalized Method

CGIT, 6/30/2008



Town of Blacksburg Curve Number: Detailed Method

CGIT, 6/30/2008



Town of Blacksburg Curve Number: NLCD Method

CGIT, 6/30/2008

Appendix E: NOAA 24-hour Precipitation Depths

- **E-1: Rainfall Depths for SCS, 24-hour design storms.**

E-1. Rainfall Depths for SCS, 24-hour design storms.

POINT PRECIPITATION
FREQUENCY ESTIMATES
FROM NOAA ATLAS 14

Virginia 37.143 N 80.365 W 2070 feet

from "Precipitation-Frequency Atlas of the United States" NOAA Atlas 14, Volume 2, Version 3
G.M. Bonnin, D. Martin, B. Lin, T. Parzybok, M.Yekta, and D. Riley
NOAA, National Weather Service, Silver Spring, Maryland, 2004

Extracted: Sun Aug 30 2009

Confidence Limits | Seasonality | Location Maps | Other Info. | GIS data | Maps | Docs | Re

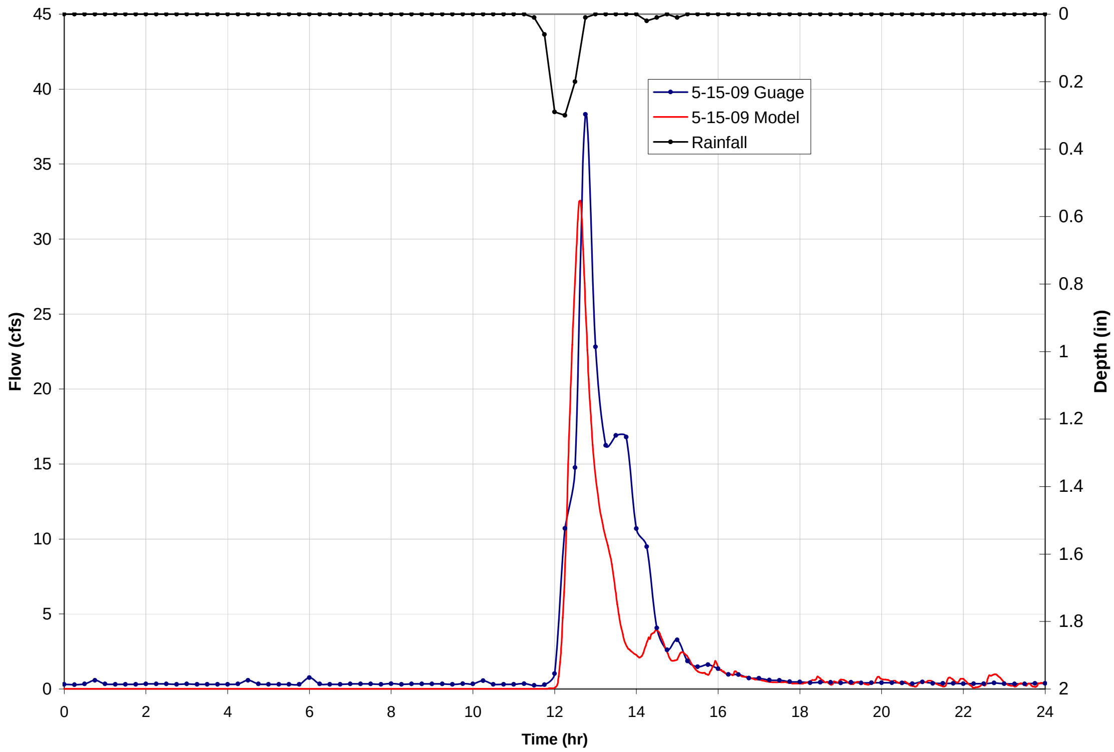
Precipitation Frequency Estimates (inches)

ARI* (years)	5 min	10 min	15 min	30 min	60 min	120 min	3 hr	6 hr	12 hr	24 hr	48 hr	4 day	7 day	10 day	20 day	30 day	45 day	60 day
1	0.30	0.48	0.60	0.82	1.03	1.20	1.28	1.56	1.89	2.31	2.75	3.09	3.59	4.13	5.61	6.97	8.82	10.61
2	0.36	0.57	0.72	1.00	1.25	1.46	1.56	1.89	2.28	2.80	3.32	3.74	4.33	4.97	6.68	8.24	10.39	12.43
5	0.43	0.69	0.87	1.24	1.59	1.86	1.98	2.38	2.86	3.56	4.20	4.71	5.38	6.08	8.01	9.69	12.04	14.19
10	0.48	0.77	0.98	1.42	1.84	2.17	2.31	2.77	3.34	4.18	4.91	5.48	6.22	6.94	9.05	10.79	13.26	15.49
25	0.55	0.87	1.11	1.64	2.18	2.59	2.76	3.32	4.04	5.09	5.92	6.58	7.38	8.11	10.44	12.22	14.82	17.10
50	0.59	0.94	1.20	1.80	2.44	2.92	3.11	3.77	4.62	5.84	6.76	7.48	8.32	9.01	11.53	13.30	15.97	18.27
100	0.64	1.01	1.28	1.97	2.71	3.26	3.48	4.25	5.24	6.66	7.65	8.44	9.29	9.93	12.62	14.35	17.05	19.34
200	0.68	1.08	1.36	2.12	2.97	3.60	3.85	4.74	5.92	7.54	8.60	9.45	10.31	10.86	13.71	15.37	18.09	20.34
500	0.73	1.16	1.46	2.32	3.32	4.07	4.35	5.44	6.90	8.81	9.95	10.88	11.72	12.12	15.16	16.69	19.39	21.54
1000	0.77	1.21	1.52	2.46	3.59	4.42	4.75	6.00	7.72	9.86	11.04	12.03	12.84	13.11	16.27	17.66	20.31	22.37

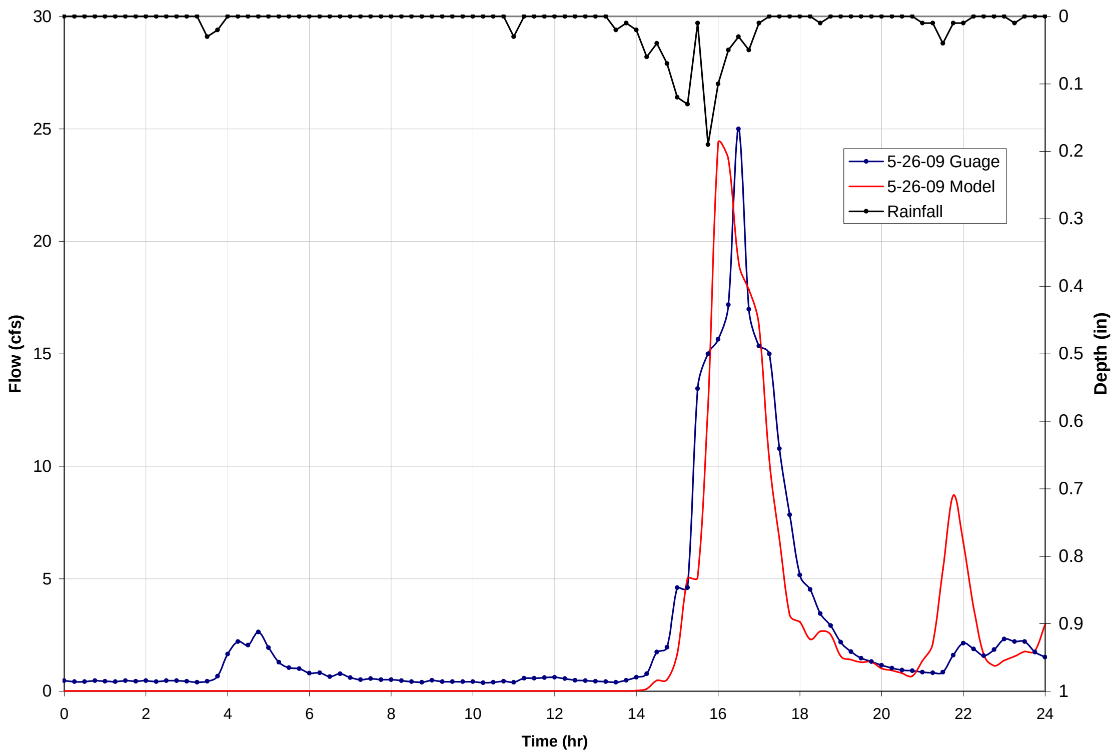
* These precipitation frequency estimates are based on a partial duration series. **ARI** is the Average Recurrence Interval.
Please refer to NOAA Atlas 14 Document for more information. NOTE: Formatting forces estimates near zero to appear as zero.

Appendix F: Comparisons between measured and modeled flow data

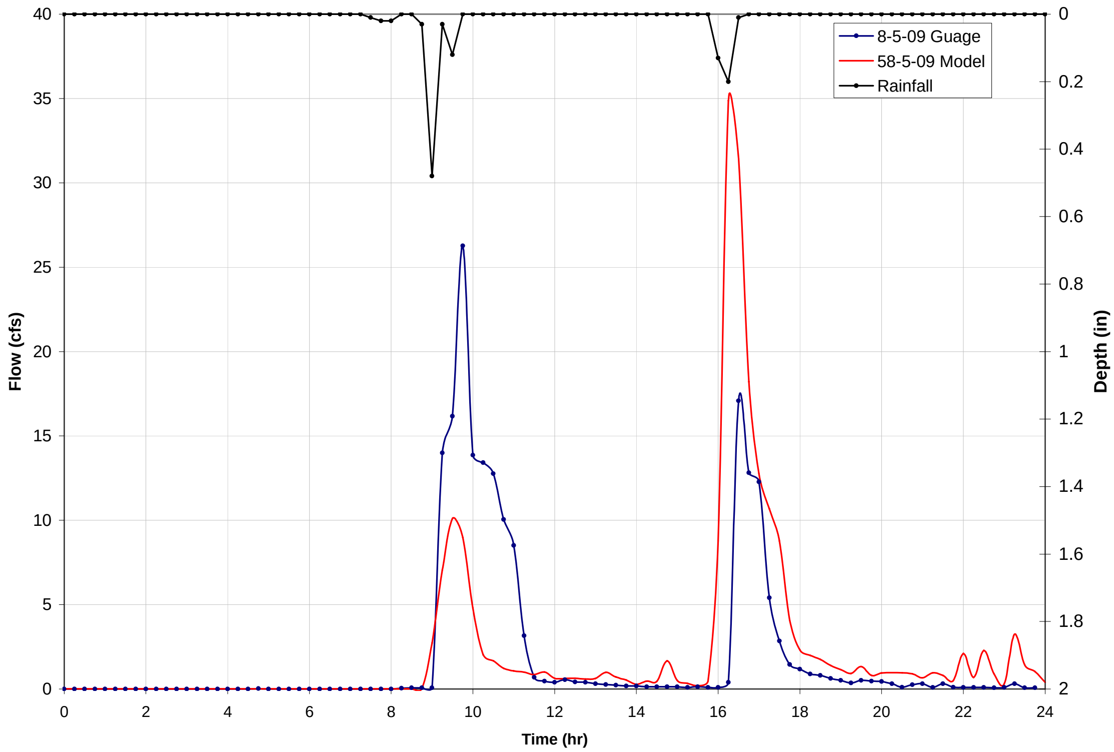
- **F-1: Comparison of measured and modeled flow data for the May 15, 2009 rain event for the subject watershed.**
- **F-2: Comparison of measured and modeled flow data for the May 26, 2009 rain event for the subject watershed.**
- **F-3: Comparison of measured and modeled flow data for the July 17, 2009 rain event for the subject watershed.**
- **F-4: Comparison of measured and modeled flow data for the August 5, 2009 rain event for the subject watershed.**



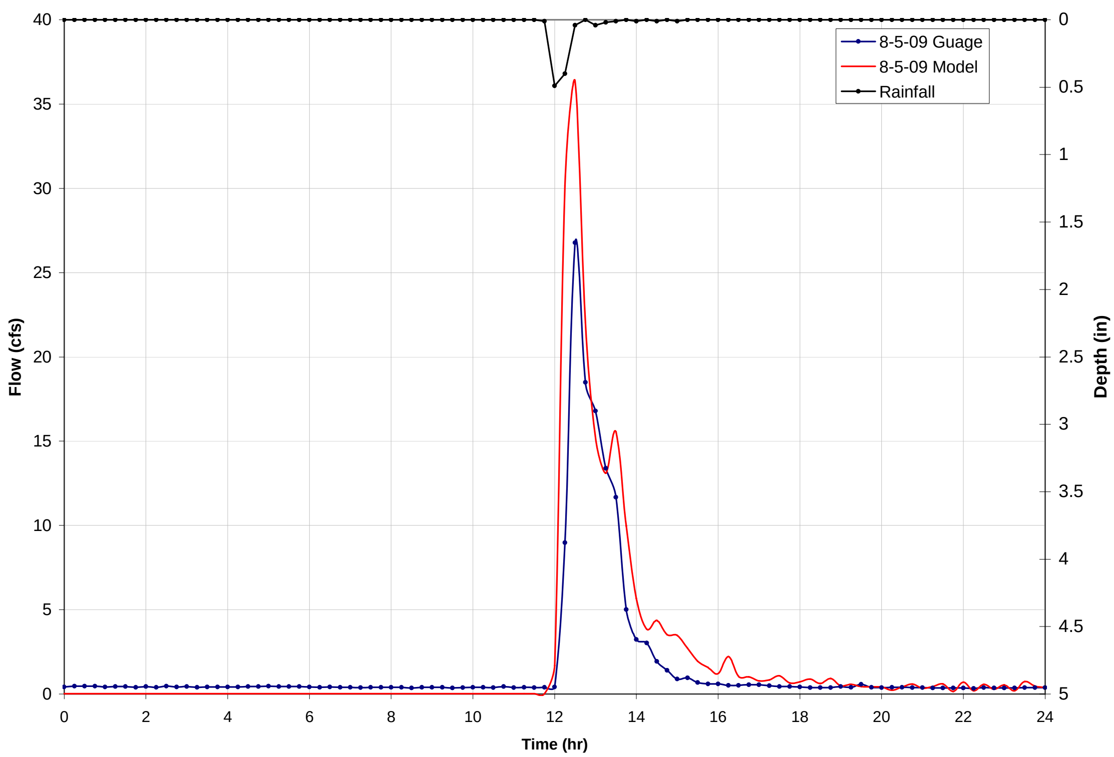
F-1. Comparison of measured and modeled flow data for the May 15, 2009 rain event for the subject watershed.



F-2. Comparison of measured and modeled flow data for the May 26, 2009 rain event for the subject watershed.



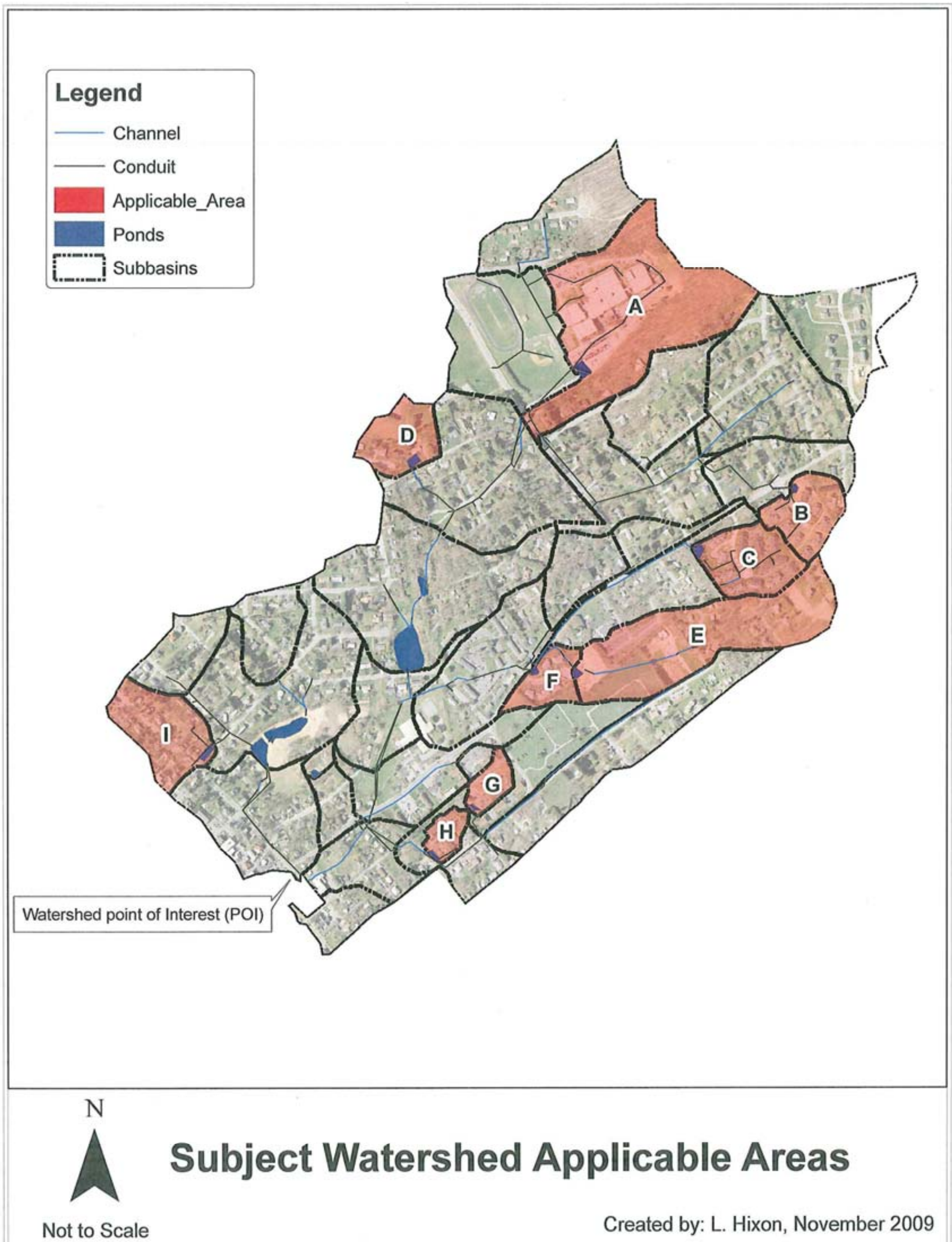
F-3. Comparison of measured and modeled flow data for the July 17, 2009 rain event for the subject watershed.



F-4. Comparison of measured and modeled flow data for the August 5, 2009 rain event for the subject watershed.

Appendix G: Mapping of Applicable Areas (A through I)

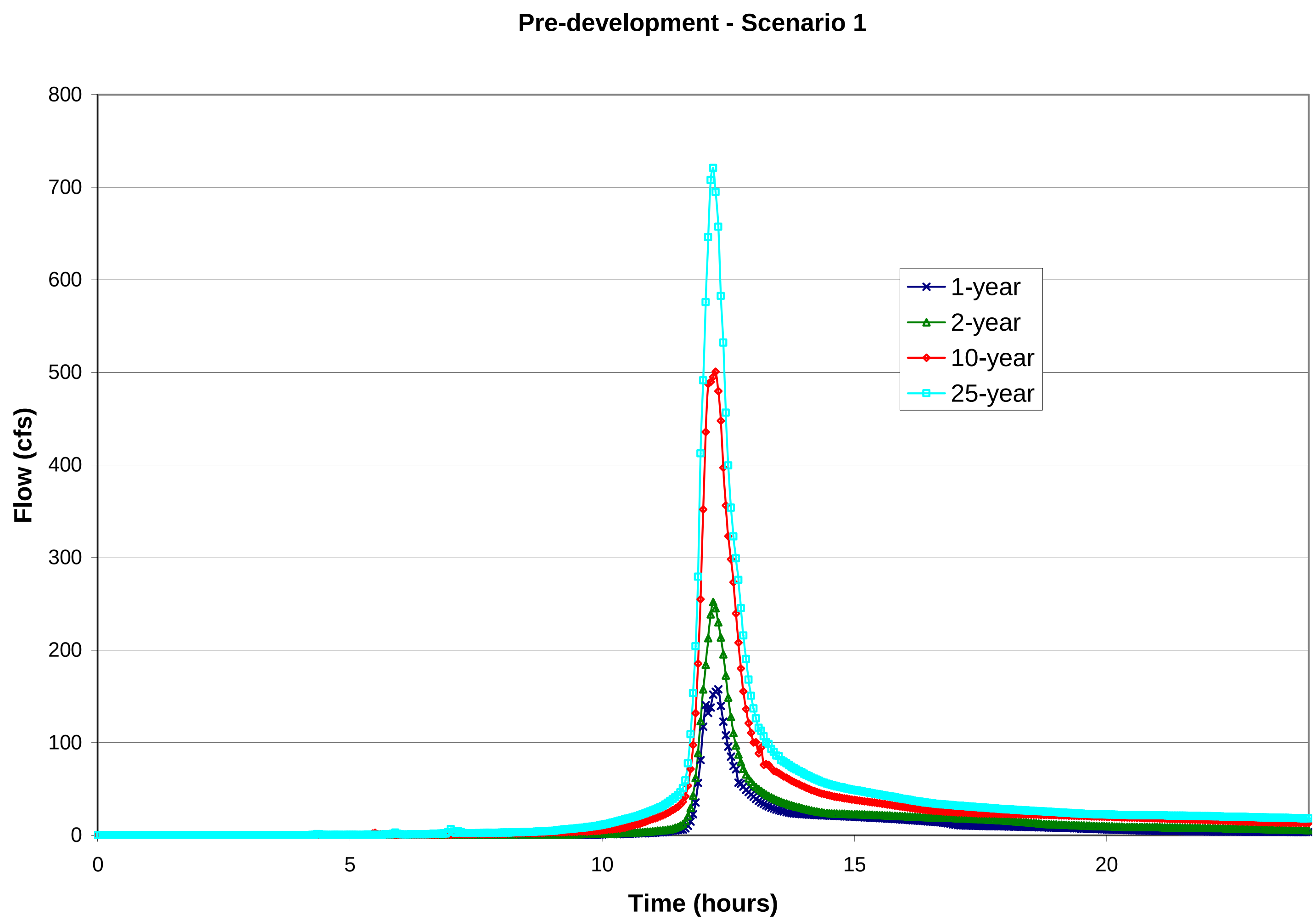
- **G-1: Map of Applicable Areas**



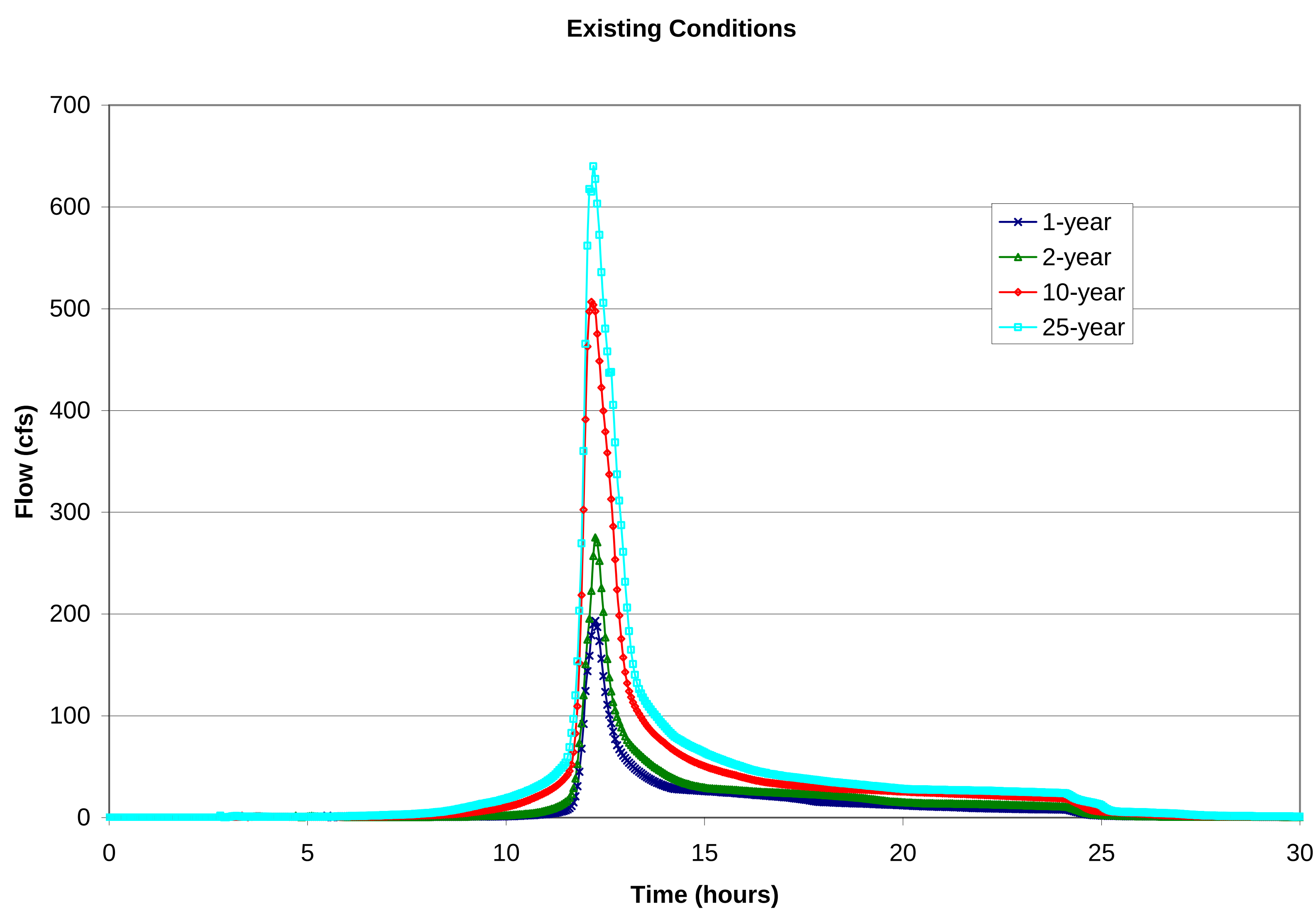
G-1. Map of Applicable Areas

Appendix H: Baseline (predevelopment), existing conditions and no detention hydrographs

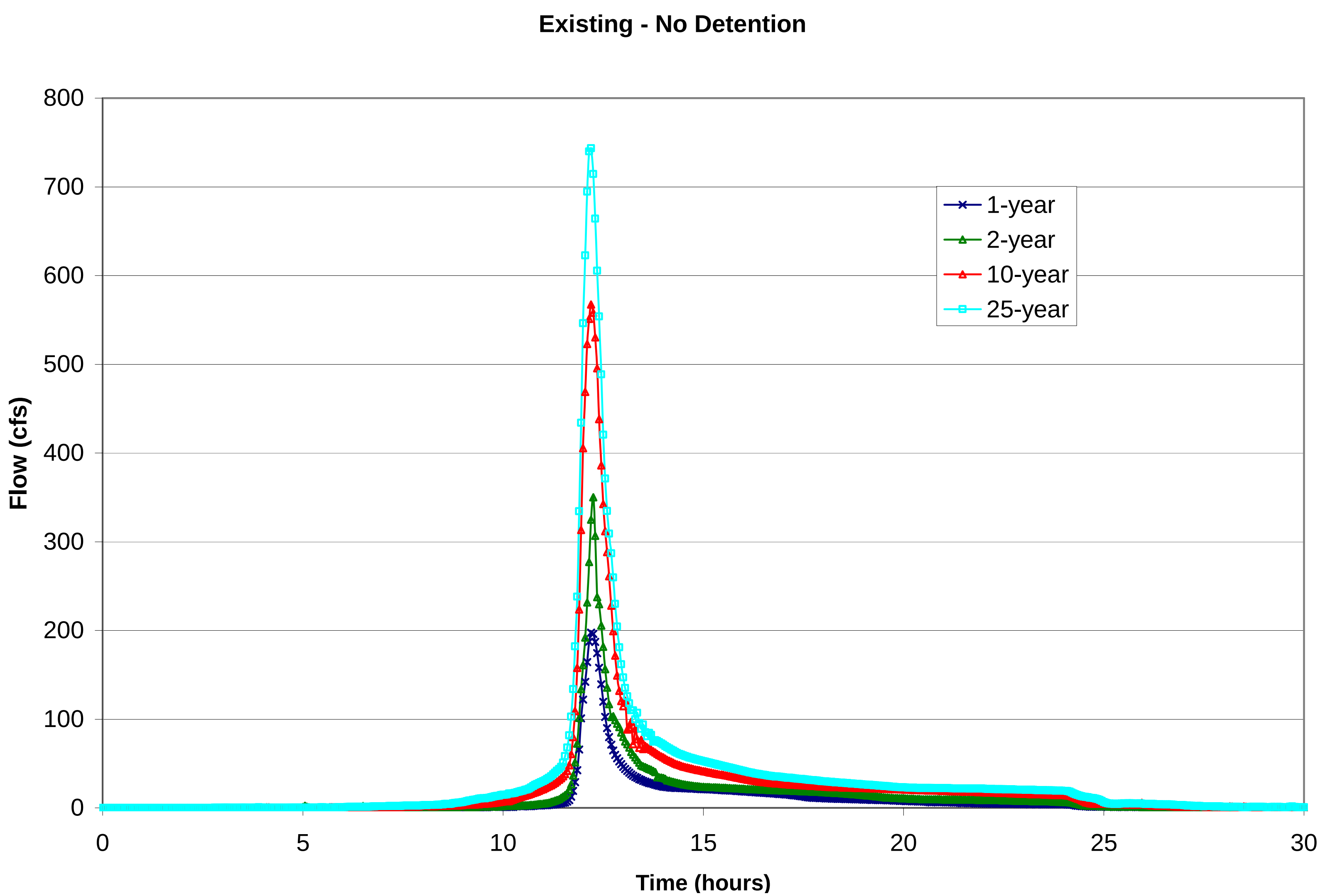
- **H-1: Predevelopment (POI baseline) hydrographs for the 1-, 2-, 10- and 25-year design storms.**
- **H-2: Existing conditions hydrographs for the 1-, 2-, 10- and 25-year design storms.**
- **H-3: Existing conditions with <u>no</u> detention hydrographs for the 1-, 2-, 10- and 25-year design storms.**
- **H-4: Hydrographs comparisons between the predevelopment, existing and existing with no detention scenarios for the 1-year storm.**
- **H-5: Hydrographs comparisons between the predevelopment, existing and existing with no detention scenarios for the 2-year storm.**
- **H-6: Hydrographs comparisons between the predevelopment, existing and existing with no detention scenarios for the 10-year storm.**
- **H-7: Hydrographs comparisons between the predevelopment, existing and existing with no detention scenarios for the 25-year storm.**



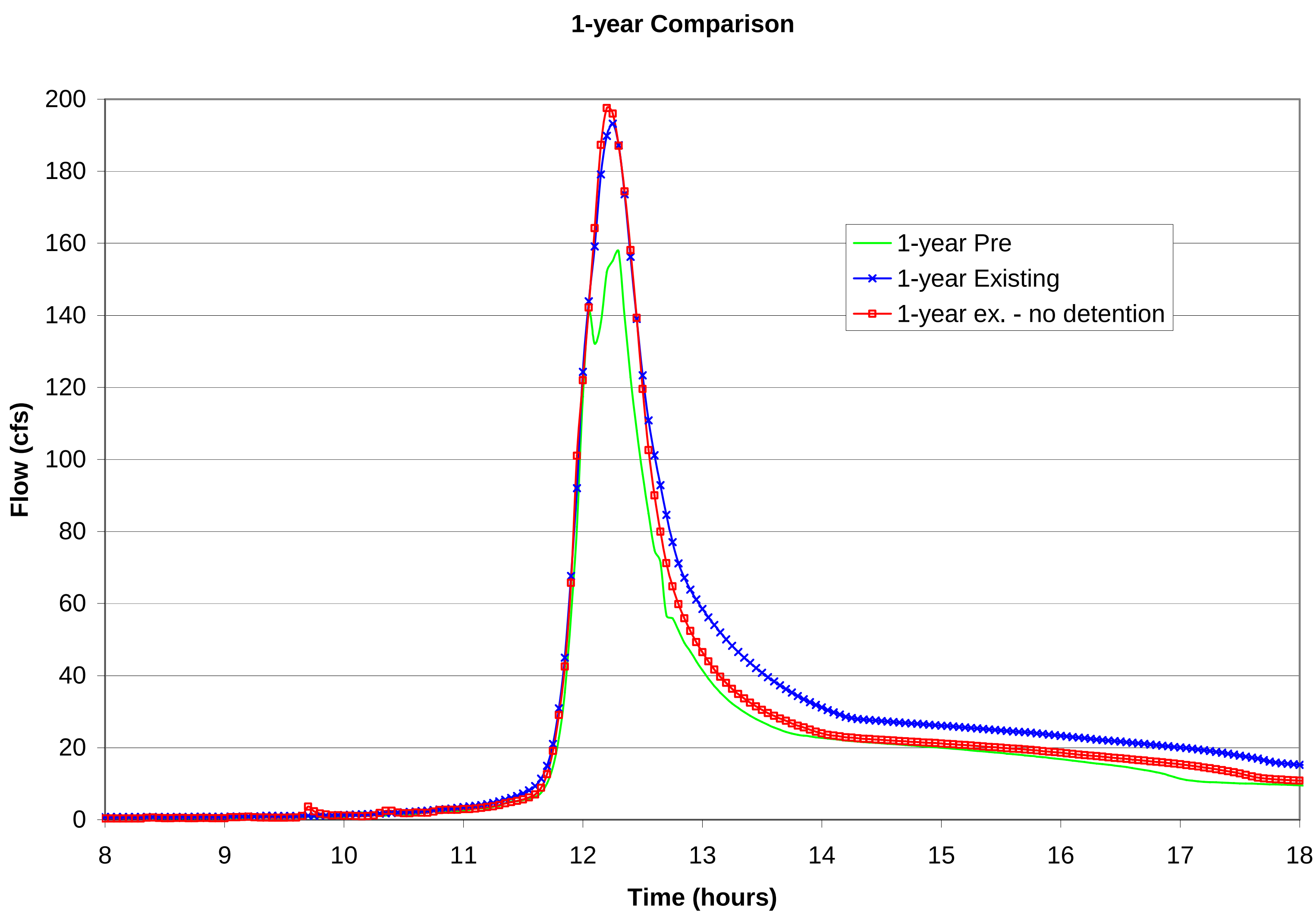
H-1. Predevelopment (POI baseline) hydrographs for the 1-, 2-, 10- and 25-year design storms.



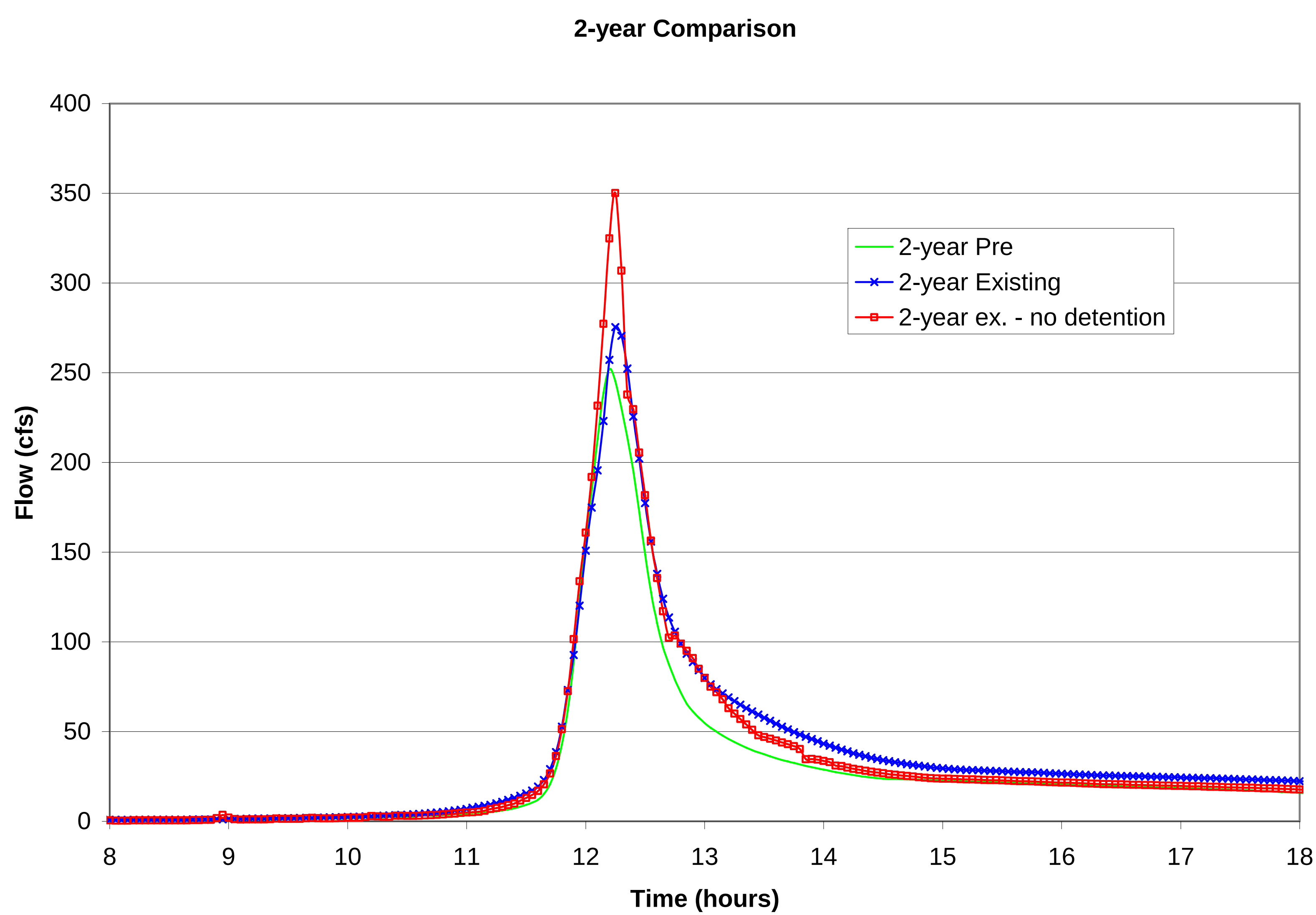
H-2. Existing conditions hydrographs for the 1-, 2-, 10- and 25-year design storms.



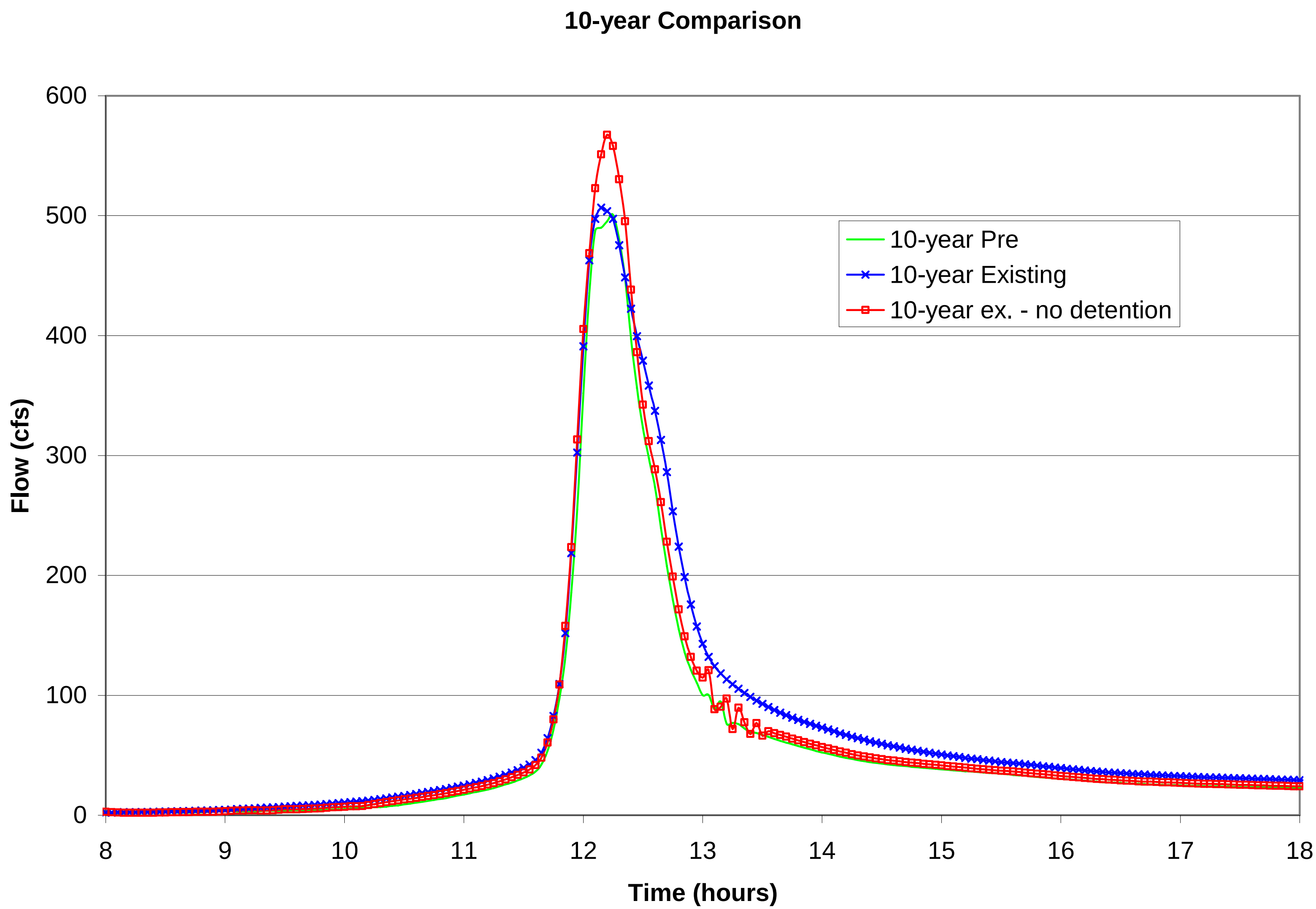
H-3. Existing conditions with <u>no</u> detention hydrographs for the 1-, 2-, 10- and 25-year design storms.



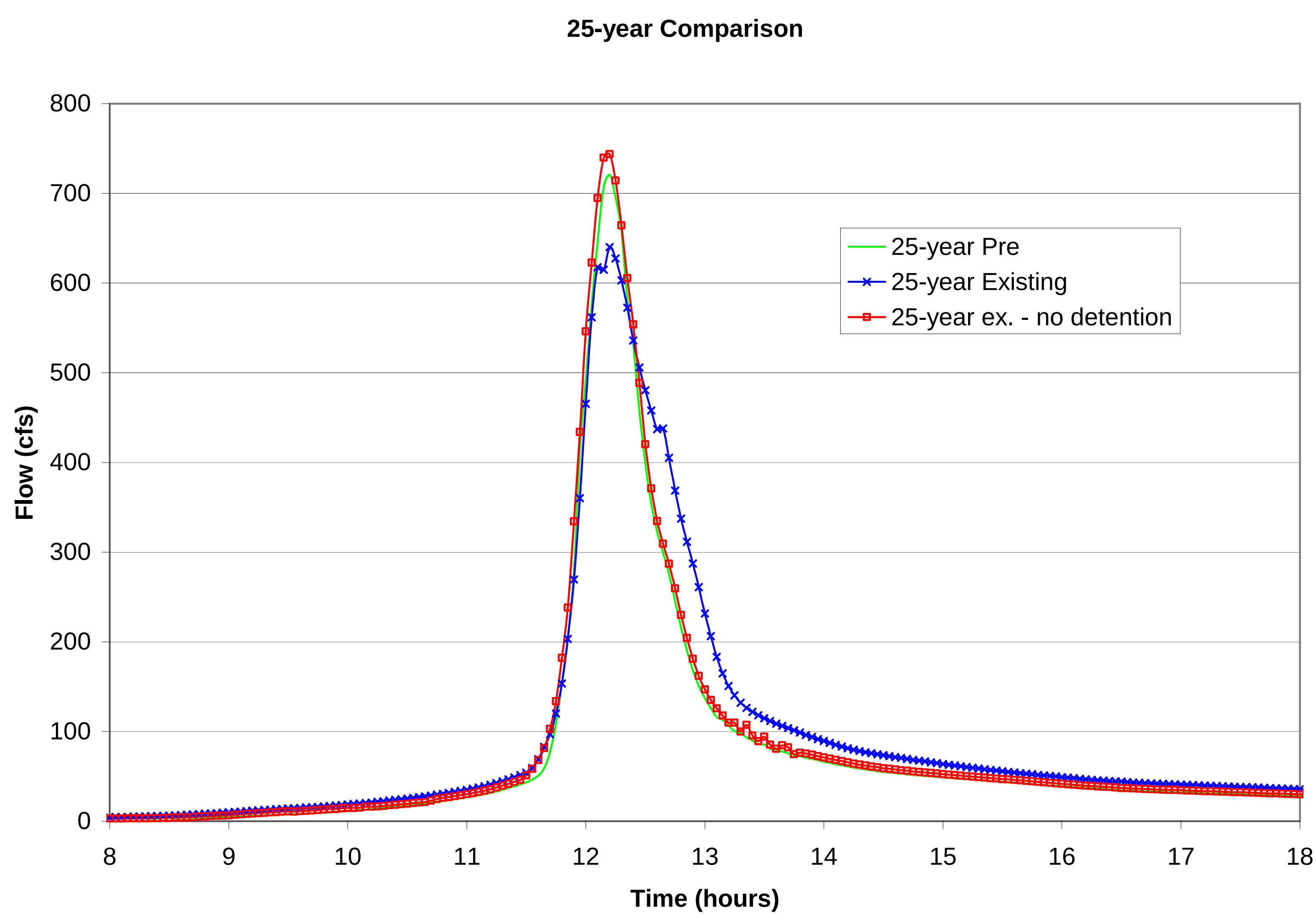
H-4. Hydrographs comparisons between the predevelopment, existing and existing with no detention scenarios for the 1-year storm.



H-5. Hydrographs comparisons between the predevelopment, existing and existing with no detention scenarios for the 2-year storm.



H-6. Hydrographs comparisons between the predevelopment, existing and existing with no detention scenarios for the 10-year storm.



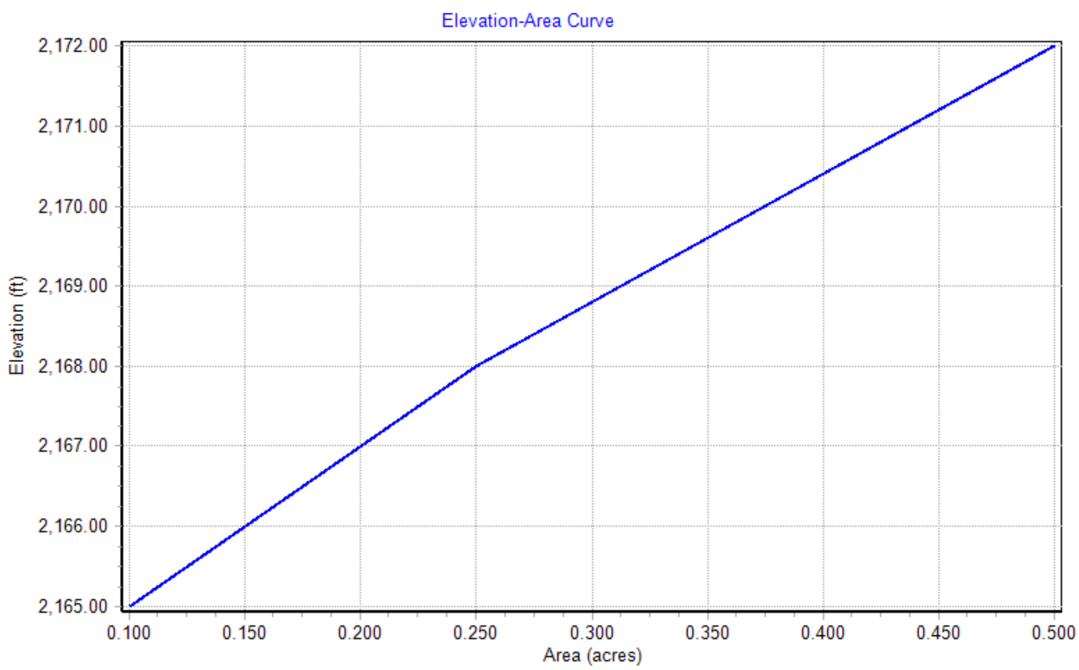
H-7. Hydrographs comparisons between the predevelopment, existing and existing with no detention scenarios for the 25-year storm.

Appendix I: Common information for modeling scenarios

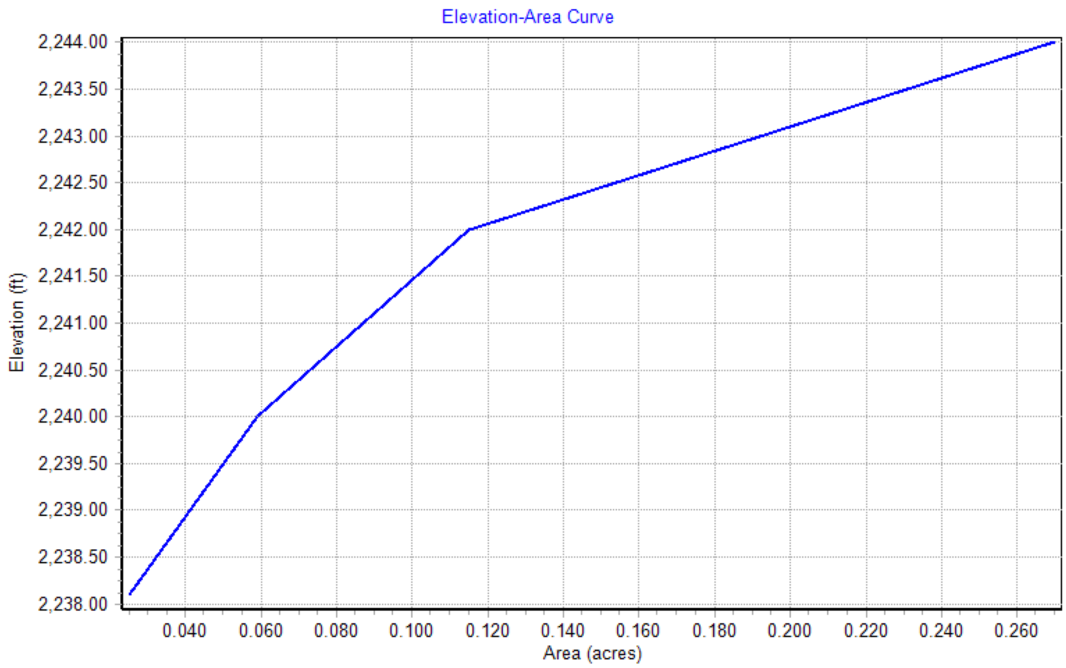
- **I-1: Pre- and post-development hydrologic parameters for applicable areas.**
- **I-2: Elevation-area information for facility serving Applicable Area A.**
- **I-3: Elevation-area information for facility serving Applicable Area B.**
- **I-4: Elevation-area information for facility serving Applicable Area C.**
- **I-5: Elevation-area information for facility serving Applicable Area D.**
- **I-6: Elevation-area information for facility serving Applicable Area E.**
- **I-7: Elevation-area information for facility serving Applicable Area F.**
- **I-8: Elevation-area information for facility serving Applicable Area G.**
- **I-9: Elevation-area information for facility serving Applicable Area H.**
- **I-10: Elevation-area information for facility serving Applicable Area I.**
- **I-11: SCS, Type II, 24-hour design rainfall for the 1-year storm.**
- **I-12: SCS, Type II, 24-hour design rainfall for the 2-year storm.**
- **I-13: SCS, Type II, 24-hour design rainfall for the 10-year storm.**
- **I-14: SCS, Type II, 24-hour design rainfall for the 25-year storm.**

I-1. Pre- and post-development hydrologic parameters for applicable areas.

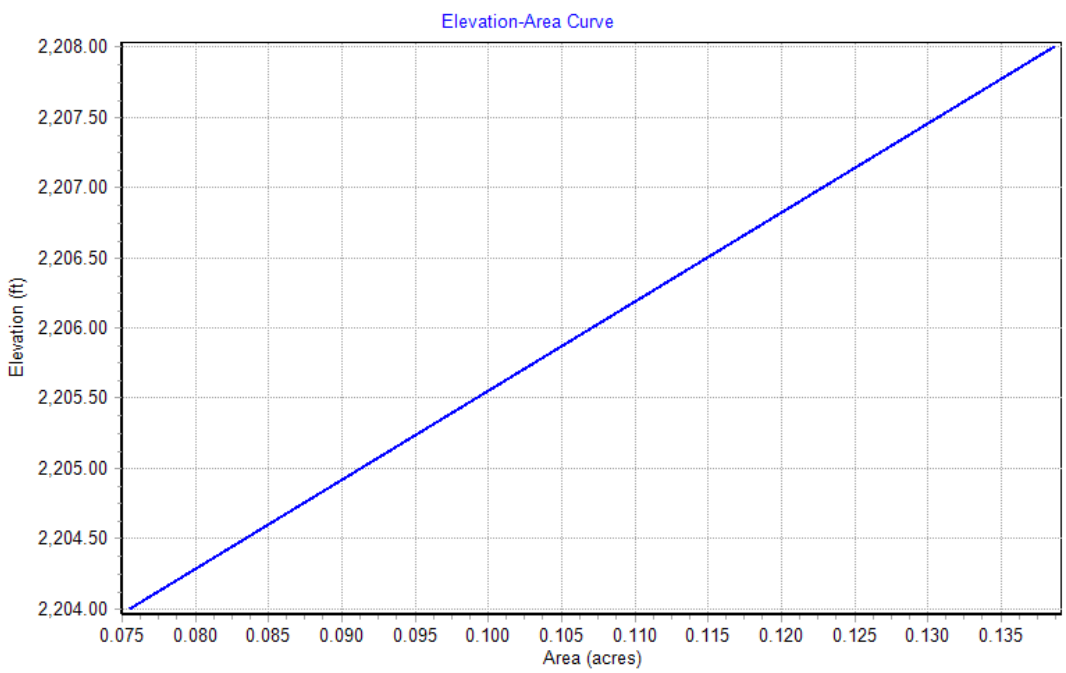
		Post-development		Pre-development	
Applicable Area	Area	CN	T_C (min)	CN	T_C (min)
A	30.27	82	19.5	71	24.5
B	5.97	84	11.0		14
C	7.56	86	7.4		9.5
D	6.05	82	13.4		14.5
E	21.53	77	17.3		21.5
F	3.98	84	17.8		12.5
G	2.35	90	12.7		15.5
H	2.07	93	9.0		12.5
I	8.77	88	13.5		15.7



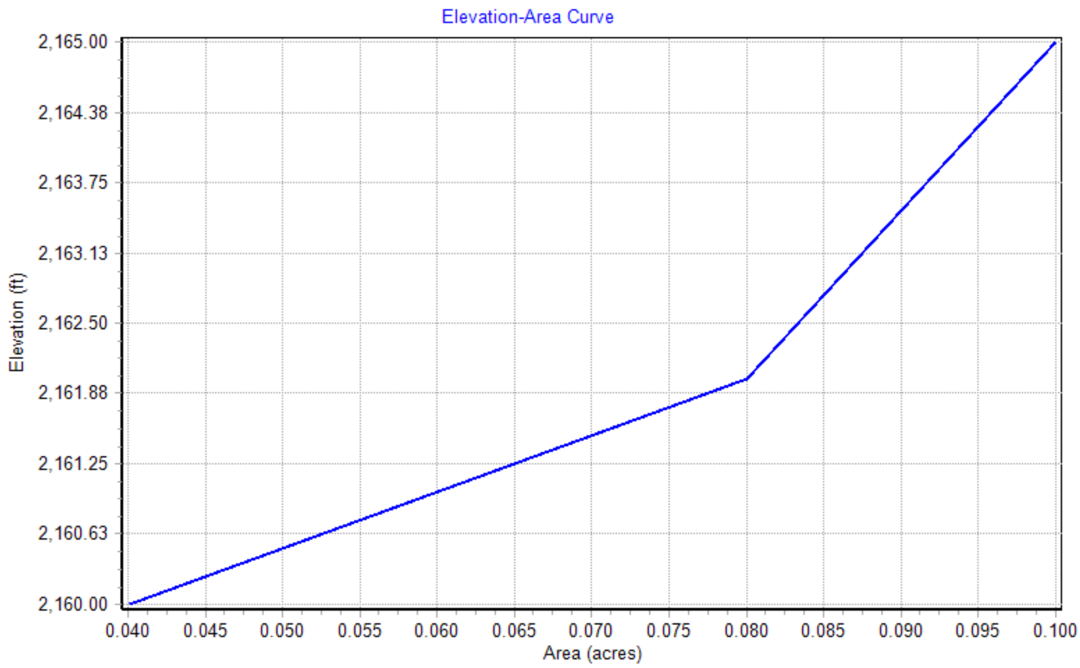
I-2. Elevation-area information for facility serving Applicable Area A.



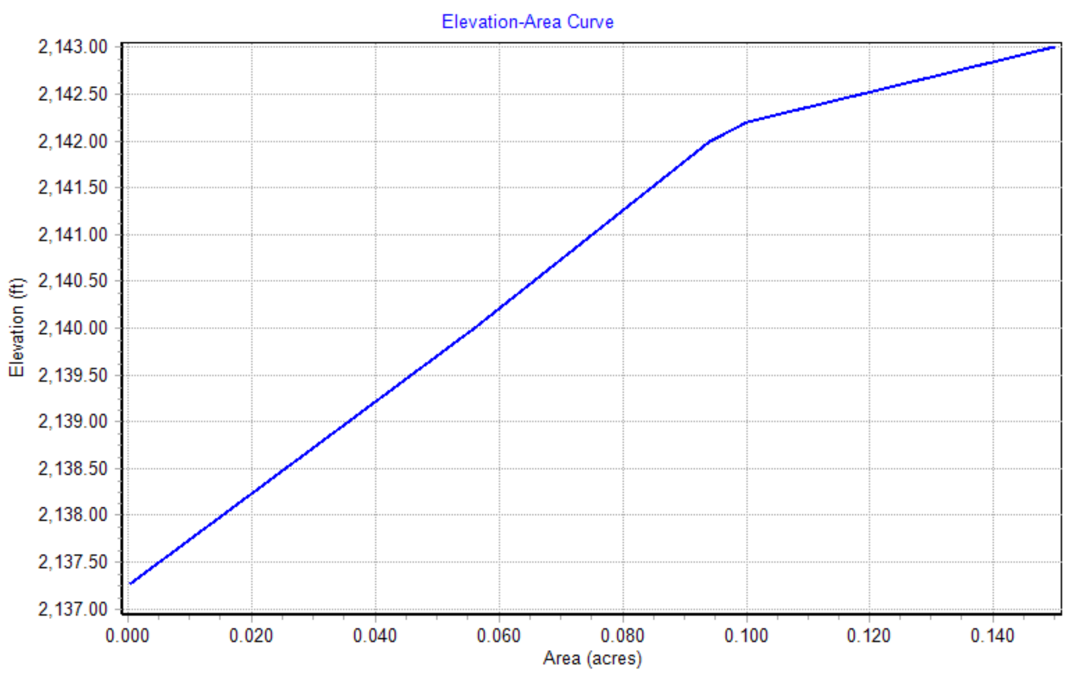
I-3. Elevation-area information for facility serving Applicable Area B.



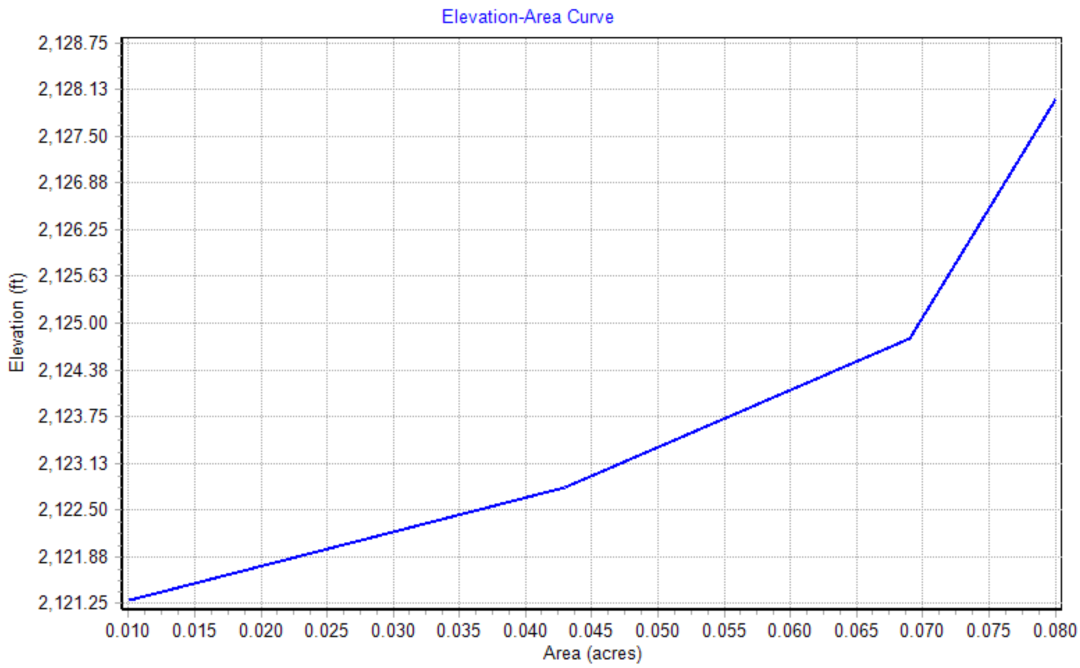
I-4. Elevation-area information for facility serving Applicable Area C.



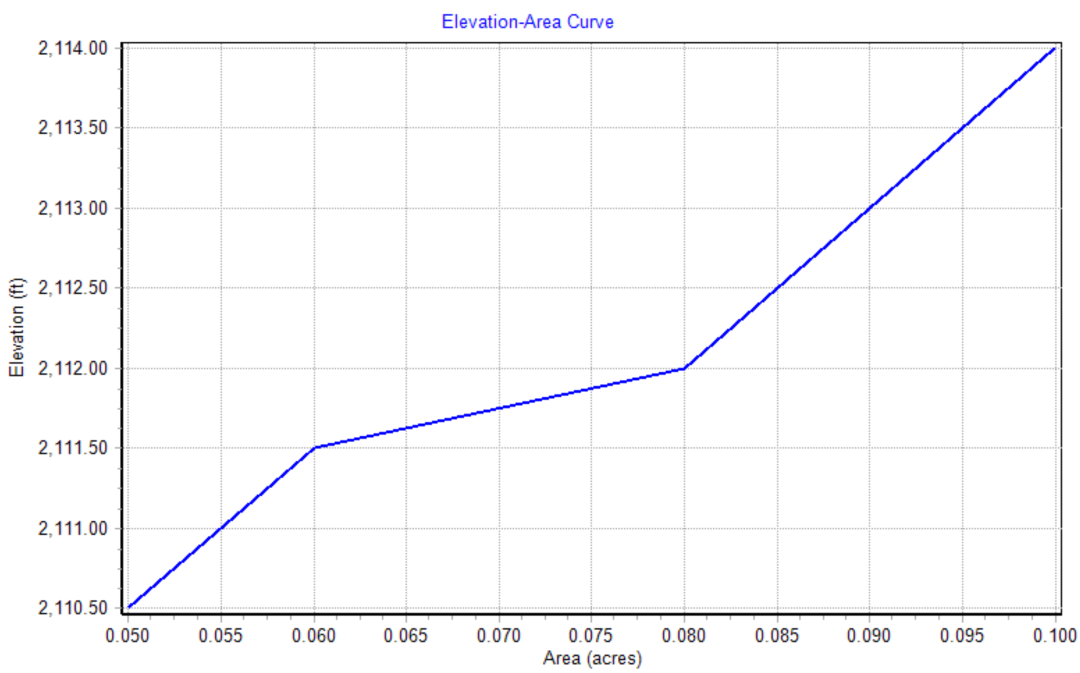
I-5. Elevation-area information for facility serving Applicable Area D.



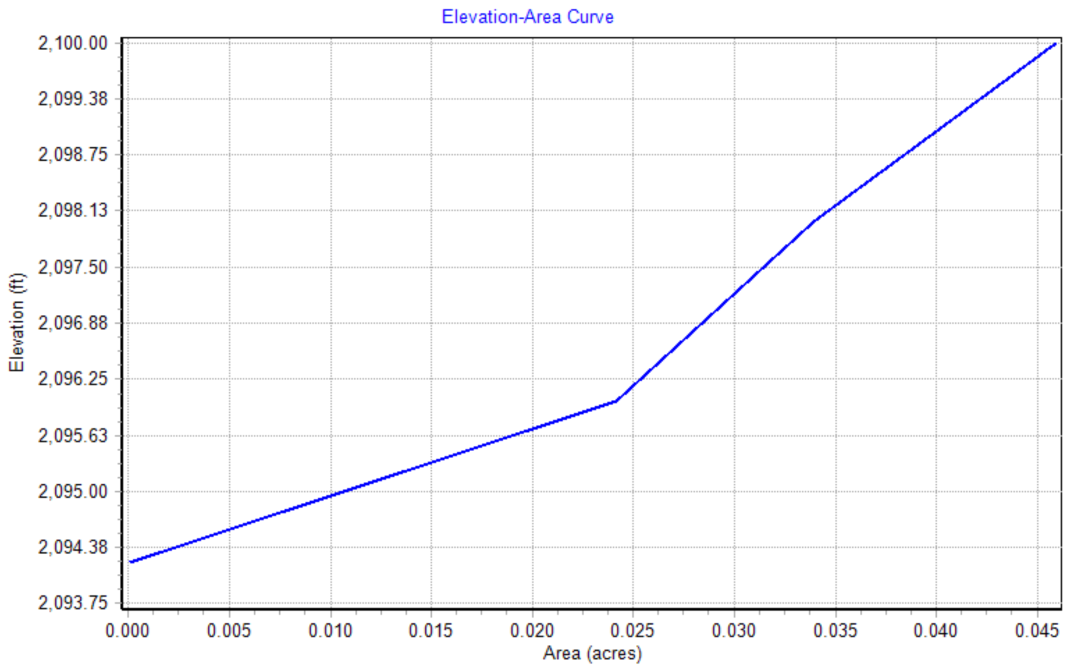
I-6. Elevation-area information for facility serving Applicable Area E.



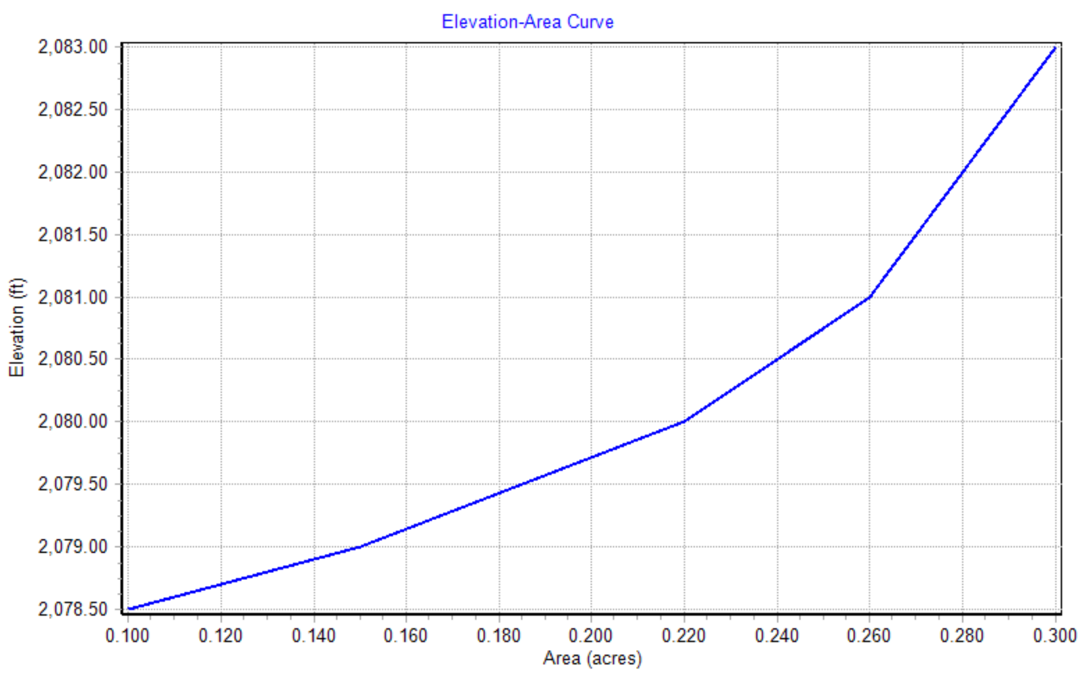
I-7. Elevation-area information for facility serving Applicable Area F.



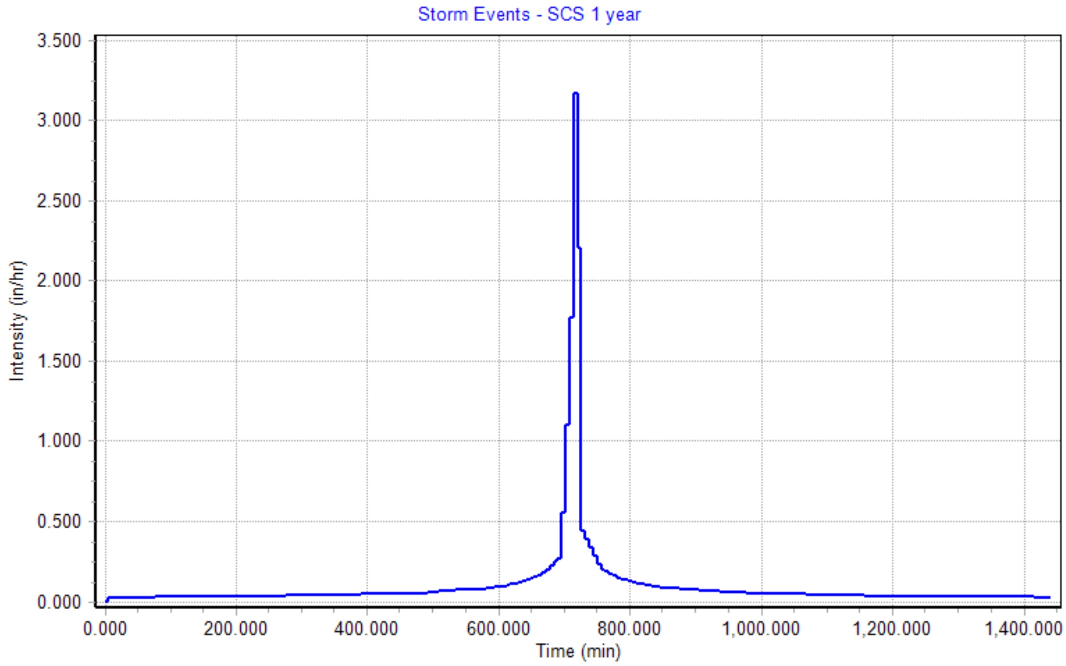
I-8. Elevation-area information for facility serving Applicable Area G.



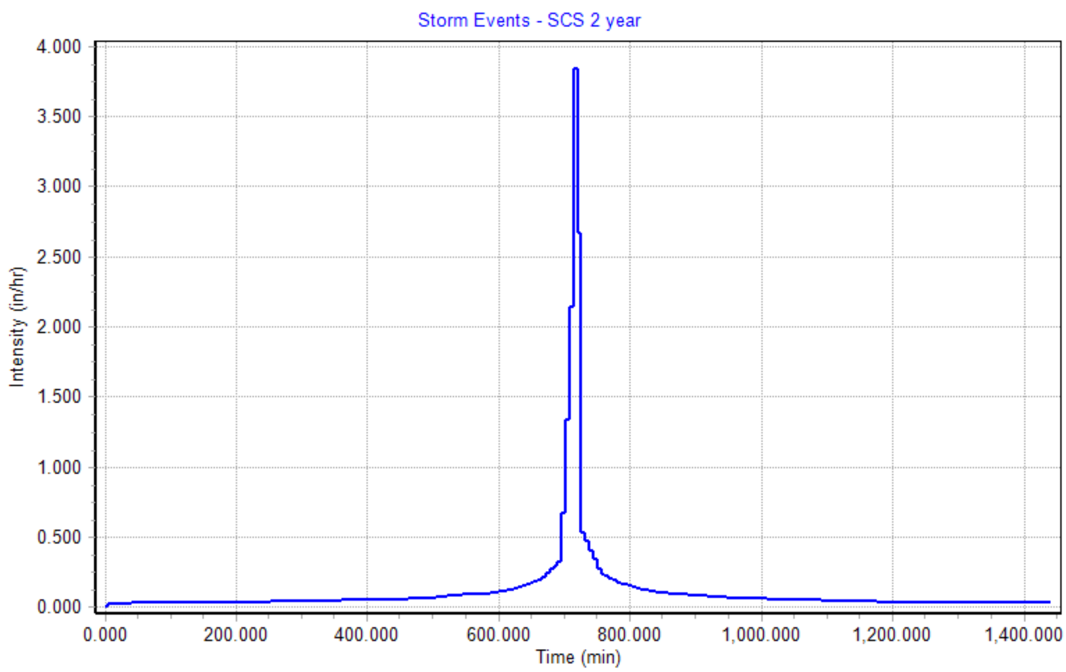
I-9. Elevation-area information for facility serving Applicable Area H.



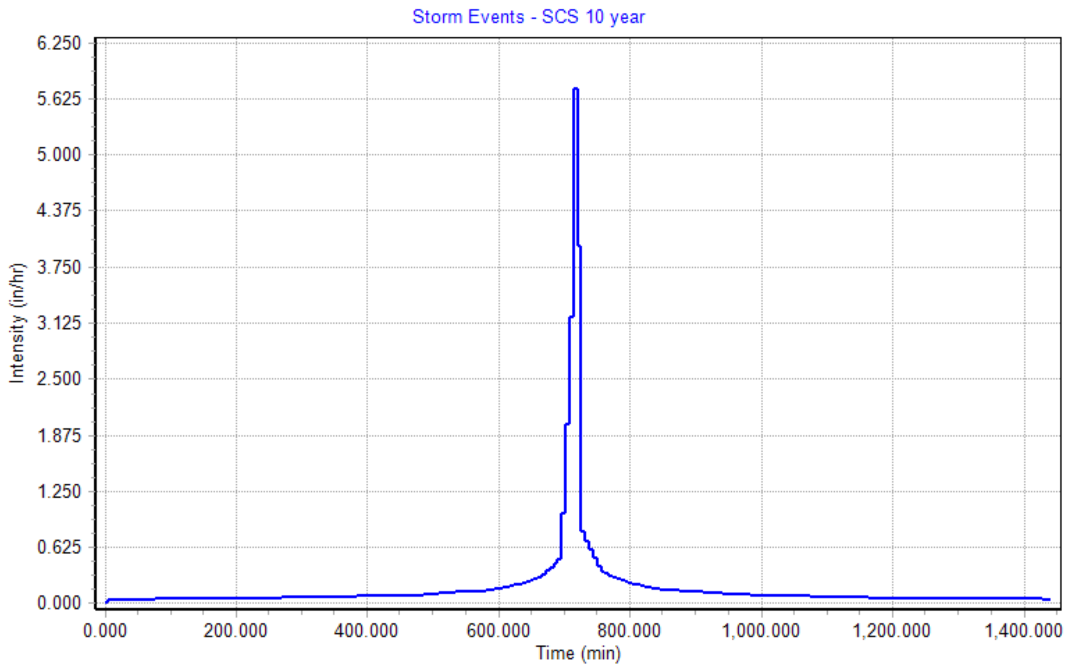
I-10. Elevation-area information for facility serving Applicable Area I.



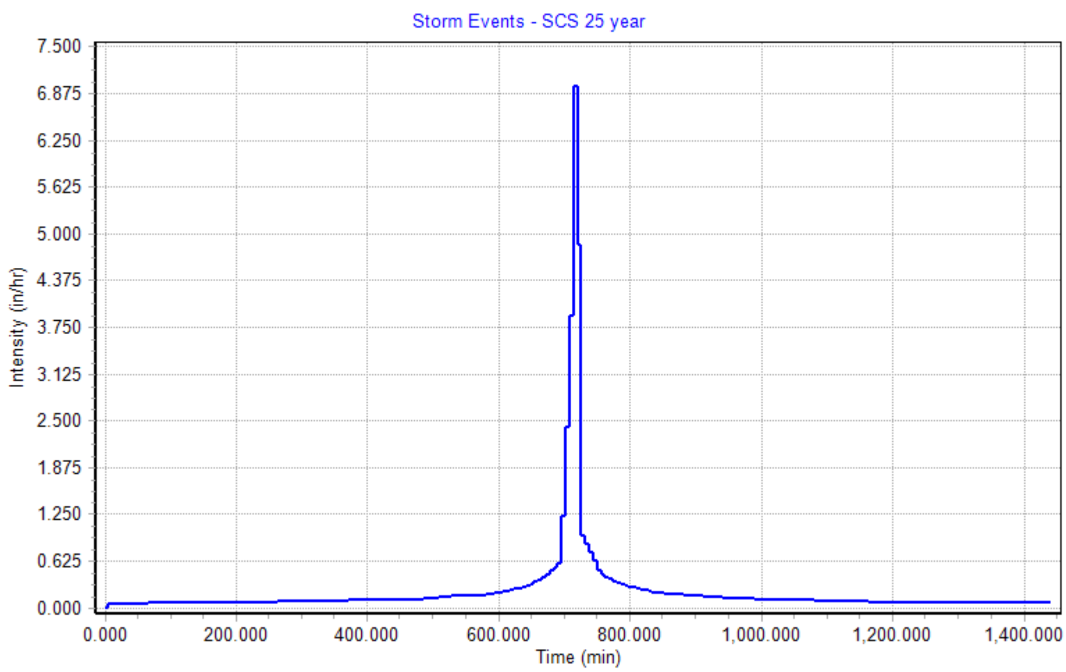
I-11. SCS, Type II, 24-hour design rainfall for the 1-year storm.



I-12. SCS, Type II, 24-hour design rainfall for the 2-year storm.



I-13. SCS, Type II, 24-hour design rainfall for the 10-year storm.



I-14. SCS, Type II, 24-hour design rainfall for the 25-year storm.

Appendix J: Information for Traditional strategy scenario

- **J-1: Traditional strategy control structure information for each facility serving each applicable area.**
- **J-2: Pre- and post-development hydrographs for applicable area A with the traditional design.**
- **J-3: Pre- and post-development hydrographs for applicable area B with the traditional design.**
- **J-4: Pre- and post-development hydrographs for applicable area C with the traditional design.**
- **J-5: Pre- and post-development hydrographs for applicable area D with the traditional design.**
- **J-6: Pre- and post-development hydrographs for applicable area E with the traditional design.**
- **J-7: Pre- and post-development hydrographs for applicable area F with the traditional design.**
- **J-8: Pre- and post-development hydrographs for applicable area G with the traditional design.**
- **J-9: Pre- and post-development hydrographs for applicable area H with the traditional design.**
- **J-10: Pre- and post-development hydrographs for applicable area I with the traditional design.**

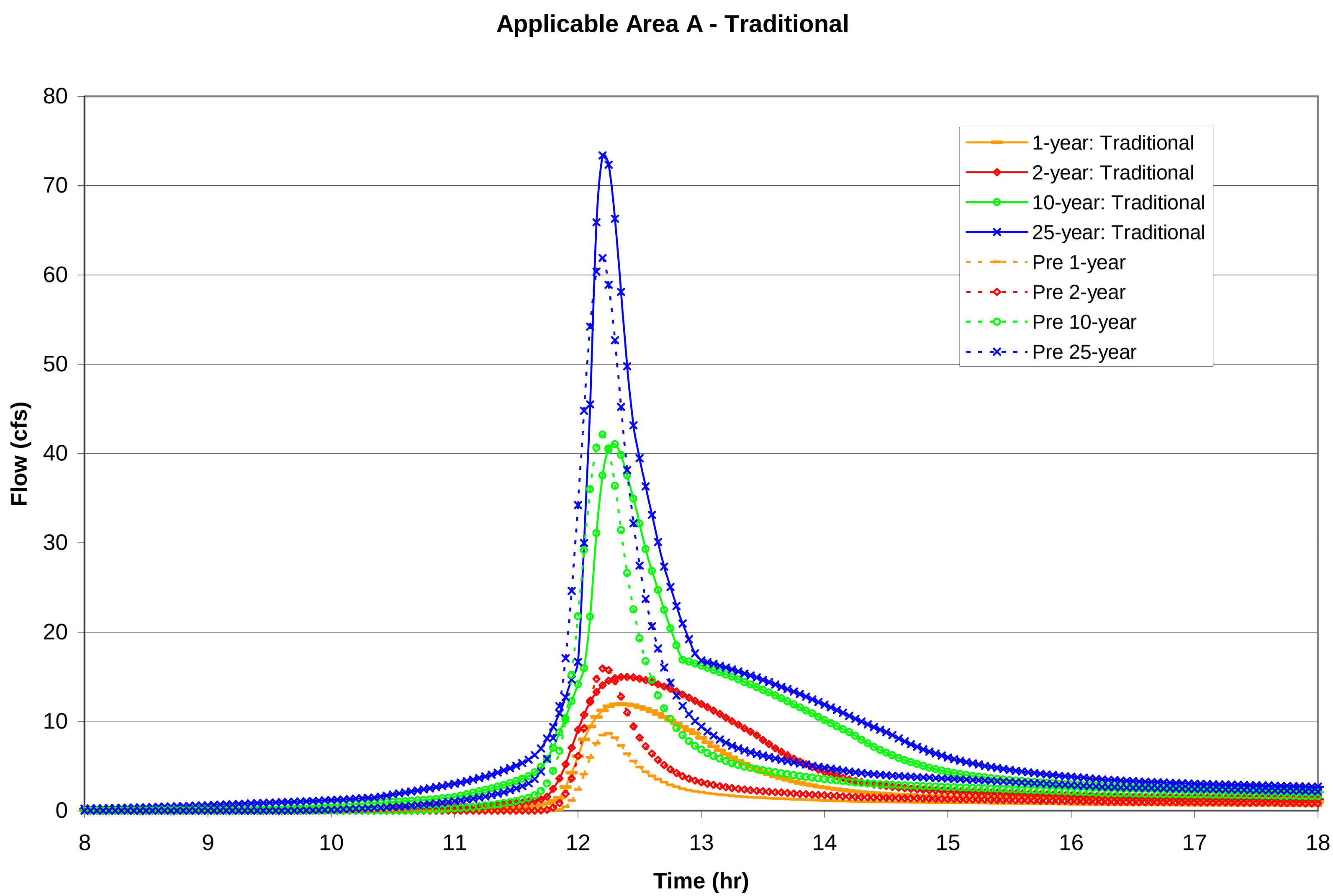
J-1. Traditional strategy control structure information for each facility serving each applicable area.

Facility Serving Applicable Area	Primary Orifice		Secondary Orifice		Riser		Weir***	
	Invert Elevation (ft)	Dia. (in)	Crest Elevation (ft)	Type/ Size* (in)	Crest Elevation (ft)	Dia (ft)	Crest Elevation (ft)	Length (ft)
A	2,165.00	19	N/A		2,170.00	30	N/A	
B	2,238.10	10	2,241.20	12	2,242.80	40	N/A	
C	2,204.00	12	N/A		2,207.10	18	2,208.50	15
D	2,160.00	12	N/A		2,162.50	24	2,163.90	15
E	2,137.26	15	N/A		2,142.30	20	2,144.50	14
F	2,121.30	7	N/A		2,124.50	12	2,125.90	30
G	2,110.50	7	N/A		2,112.70	12	2,113.30	15
H	2,094.20	3 (2)	2,099.30	10	N/A		2,100.70	6
I	2,078.50	14	2,080.80	1'H x 2'W(R)	2,081.70	40	N/A	

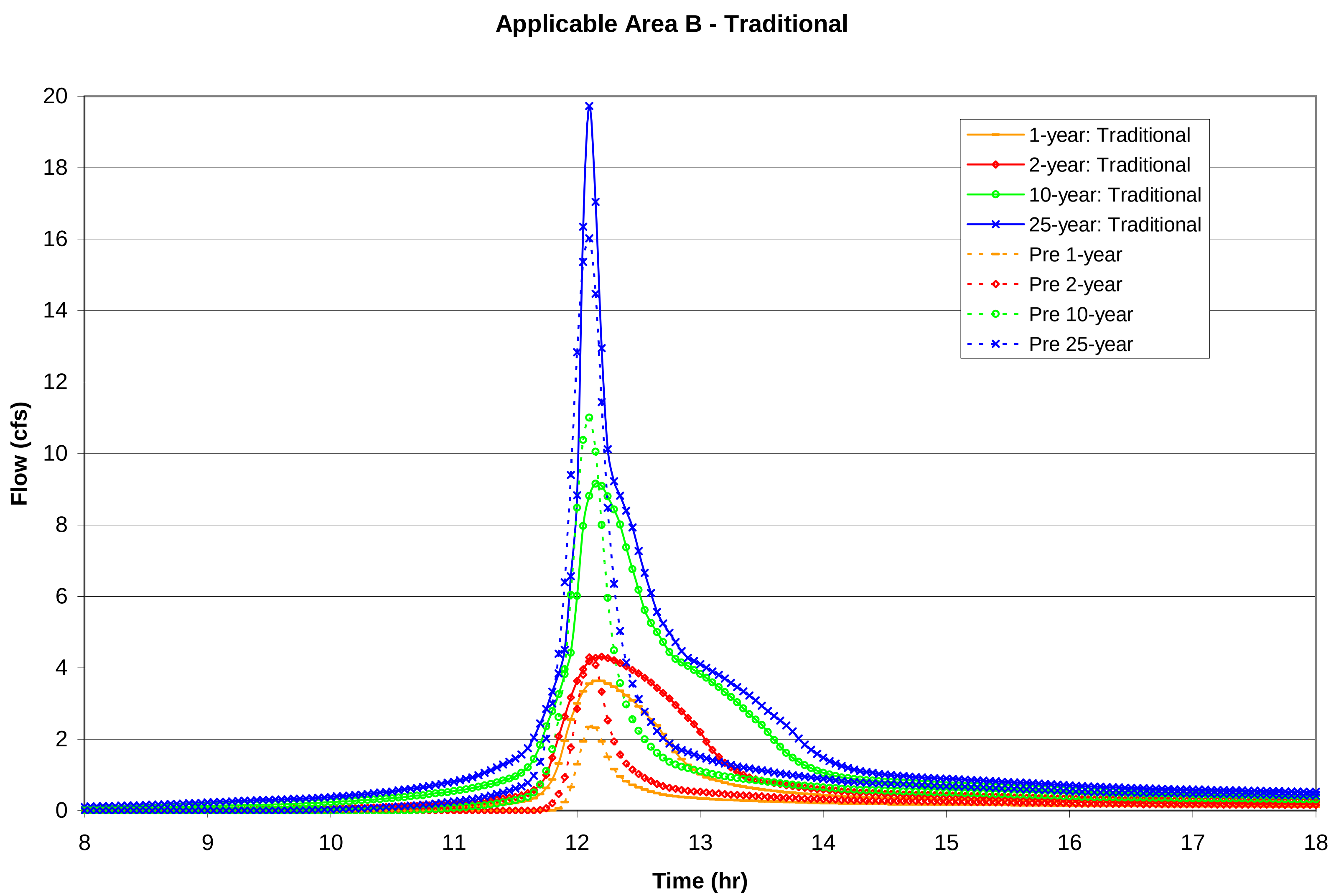
*** All orifices are circular unless an (R) is indicated for rectangular orifice, area of rectangular orifice is provided**

**** Orifice coefficient = 0.6 in all cases**

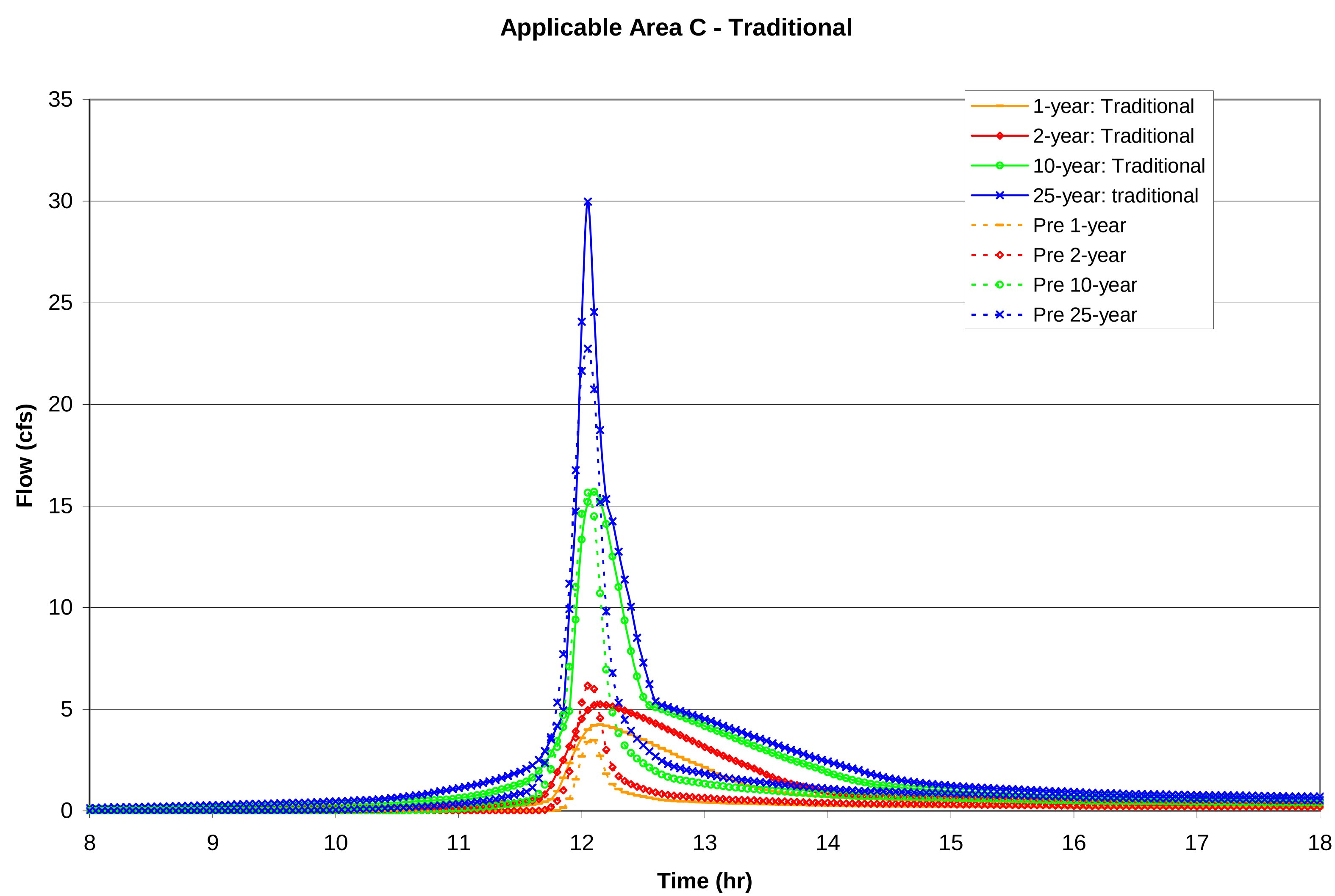
***** Weir coefficient = 3.1 in all cases; Weir is suppressed type in all cases**



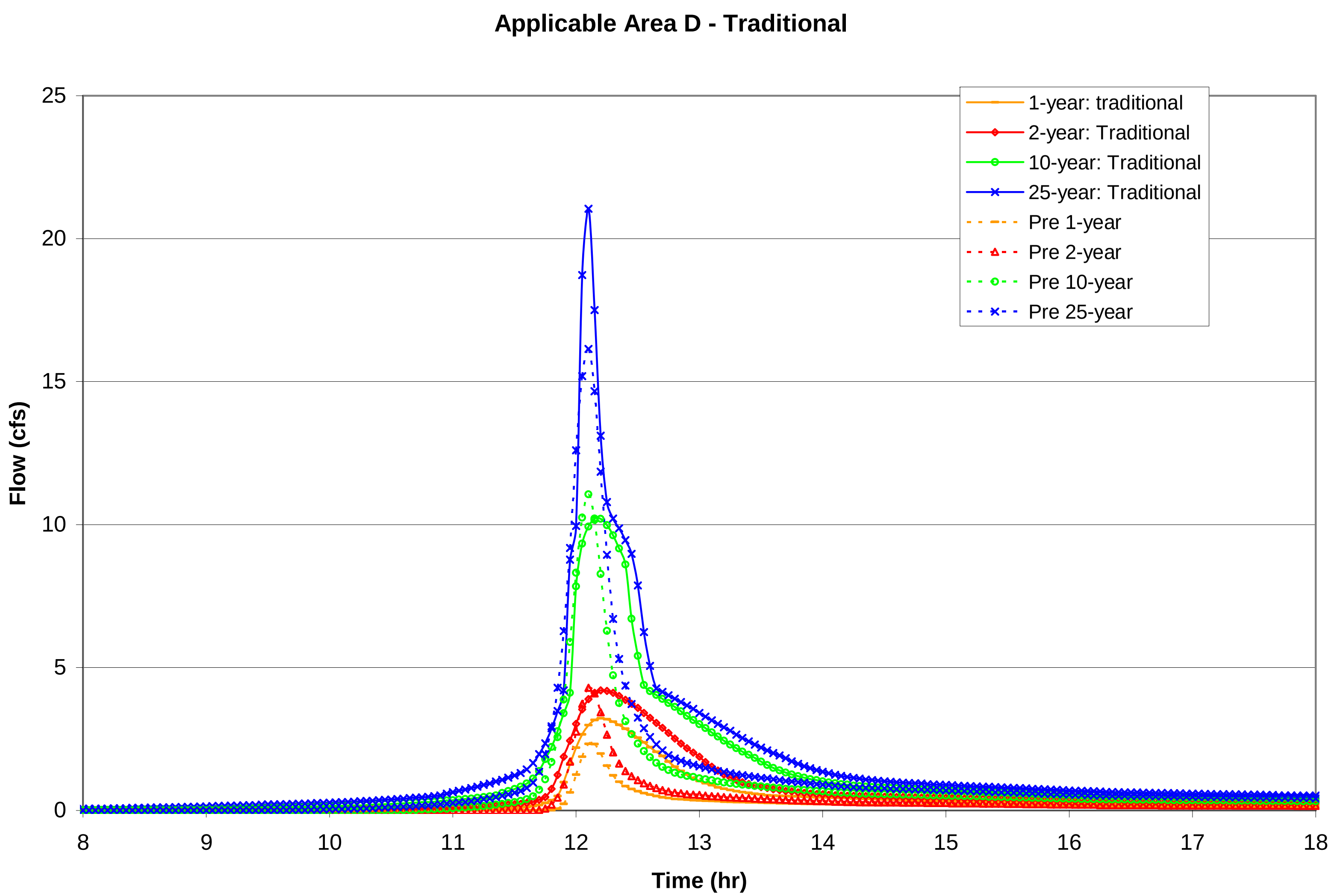
J-2. Pre- and post-development hydrographs for applicable area A with the traditional design.



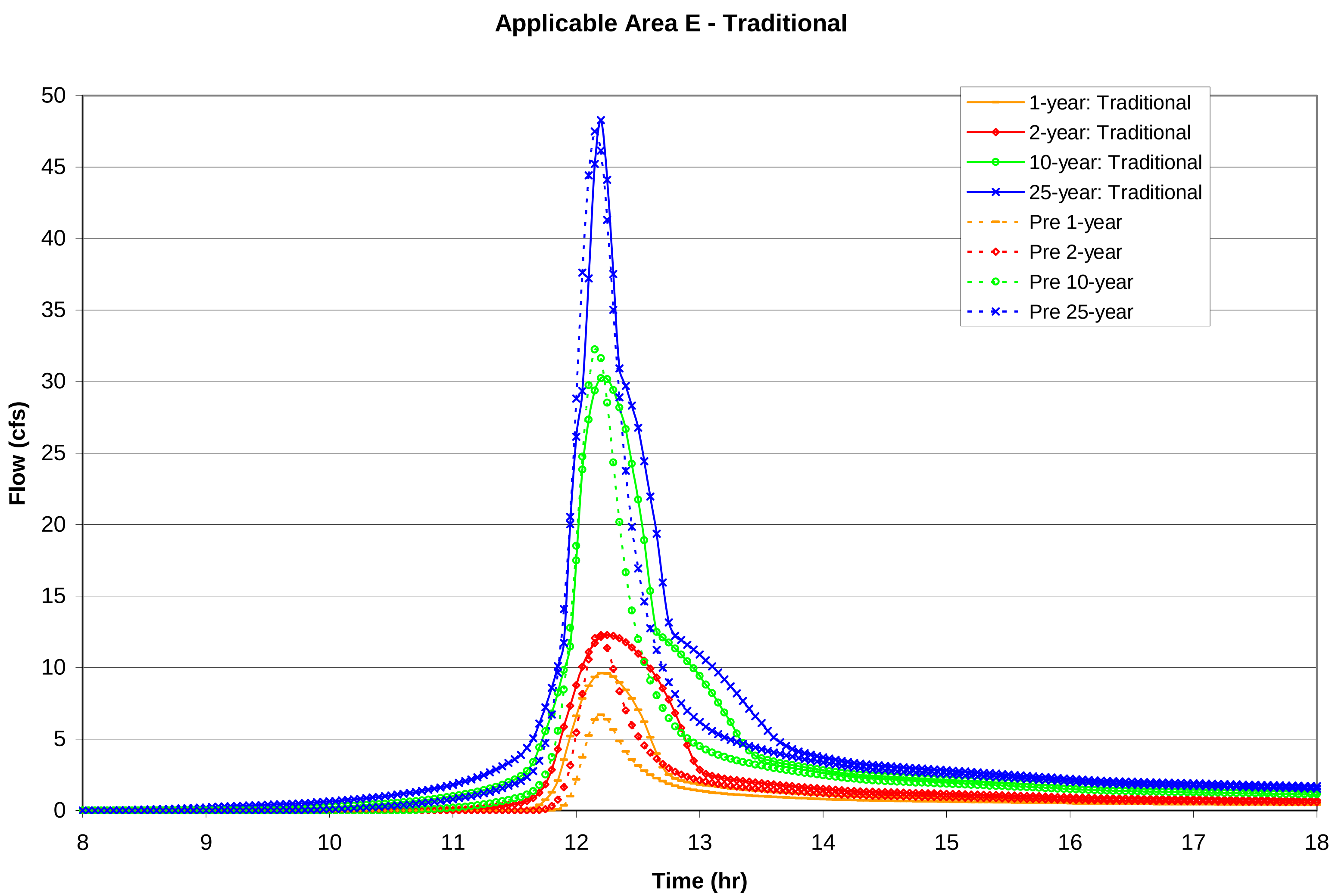
J-3. Pre- and post-development hydrographs for applicable area B with the traditional design.



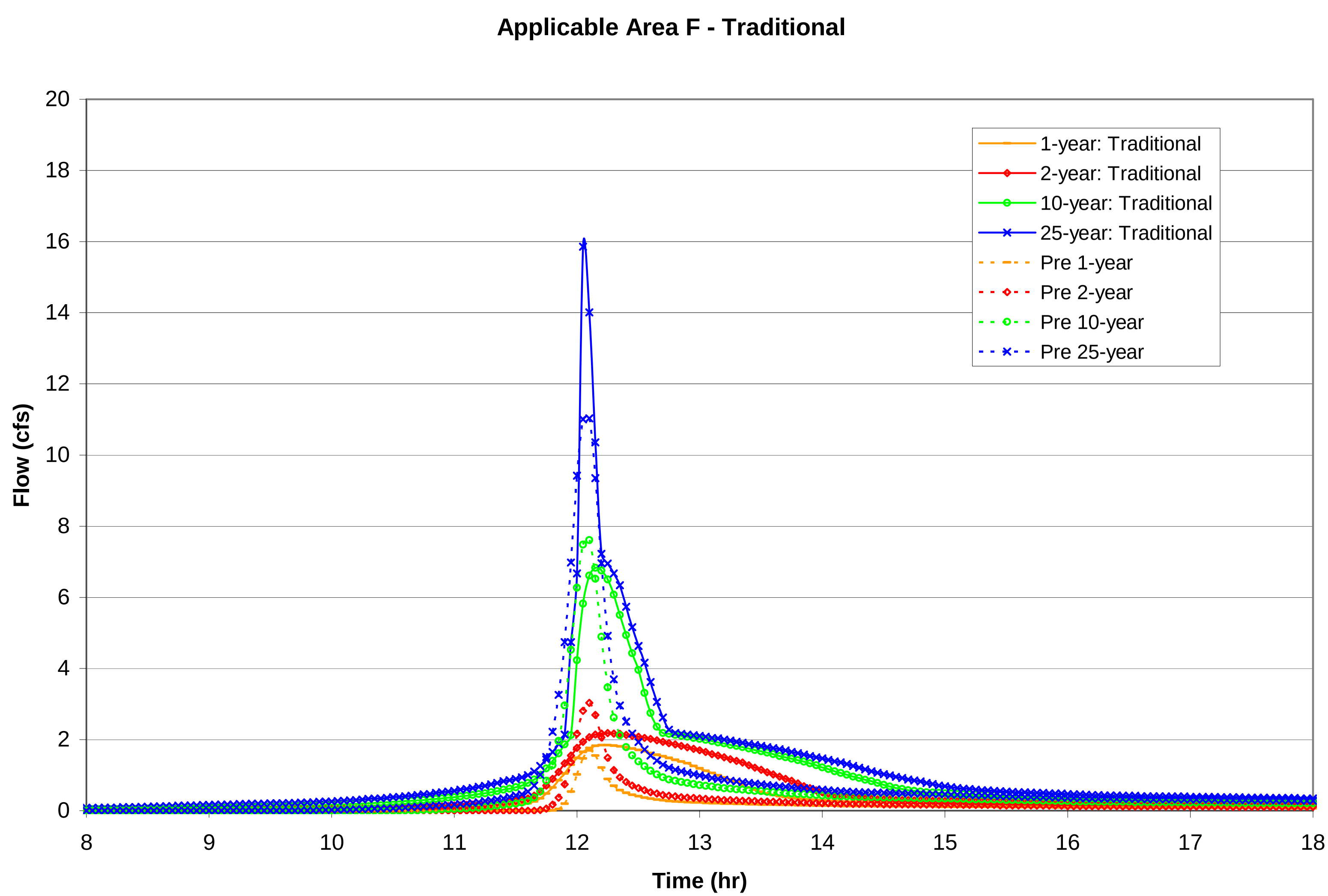
J-4. Pre- and post-development hydrographs for applicable area C with the traditional design.



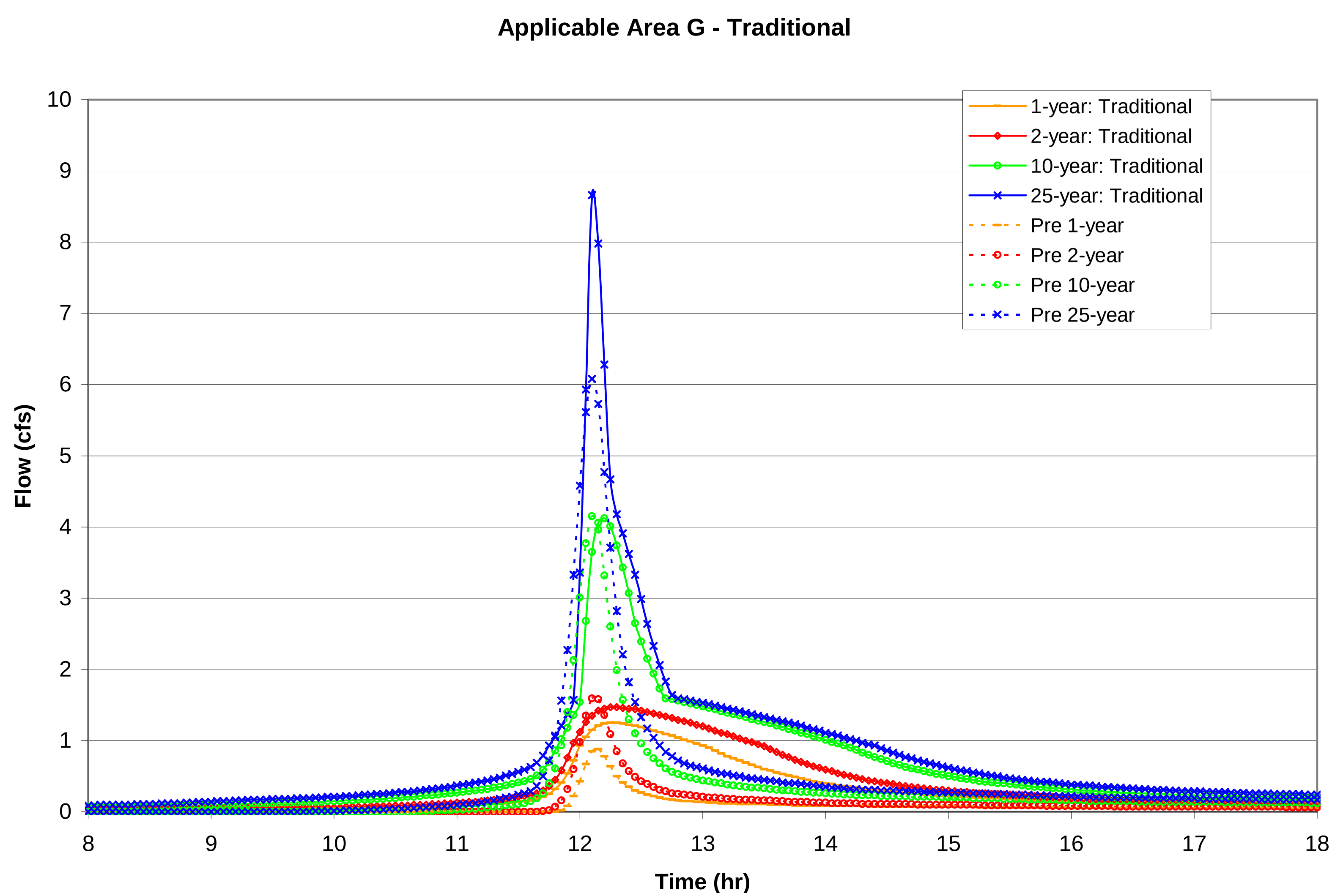
J-5. Pre- and post-development hydrographs for applicable area D with the traditional design.



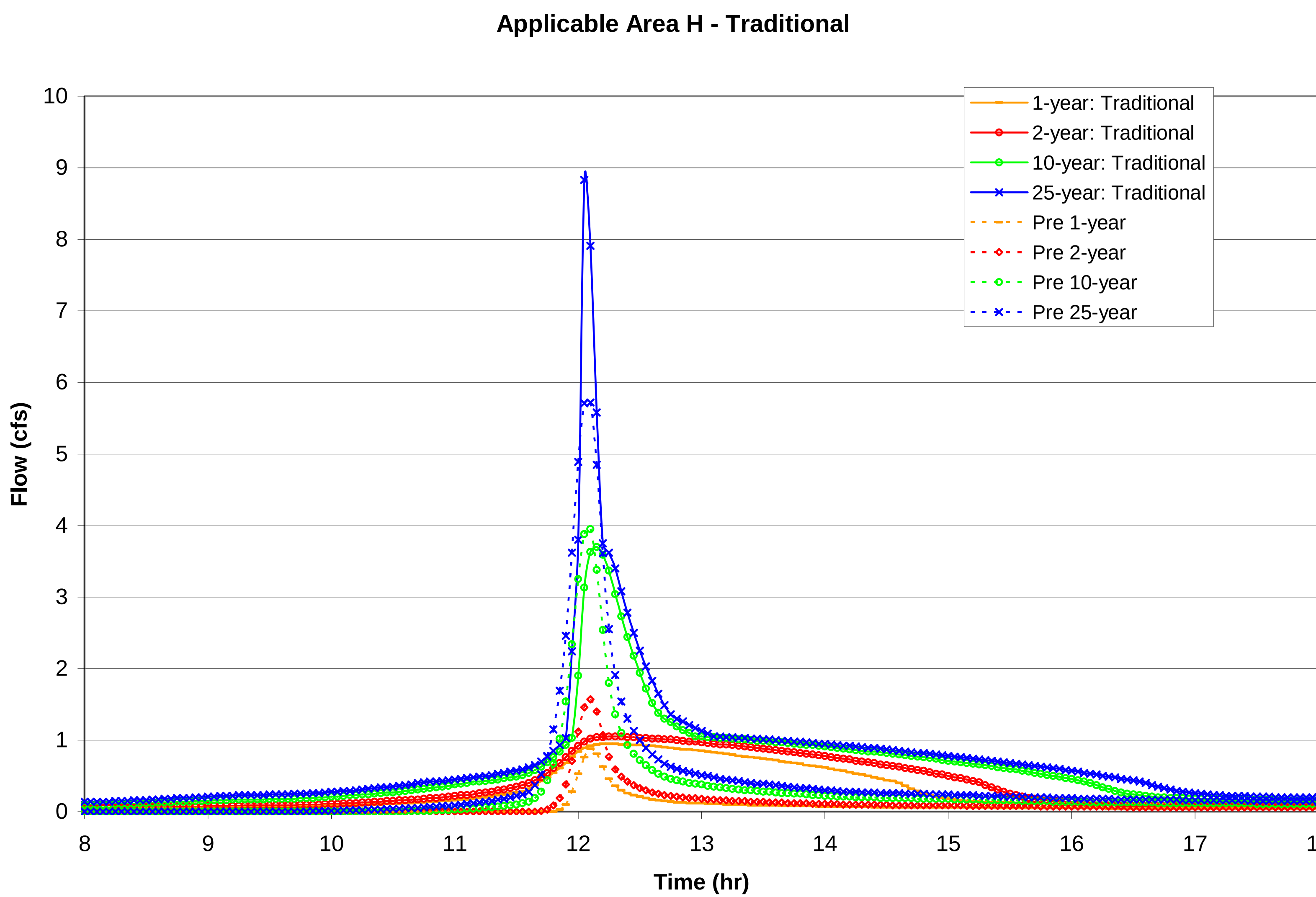
J-6. Pre- and post-development hydrographs for applicable area E with the traditional design.



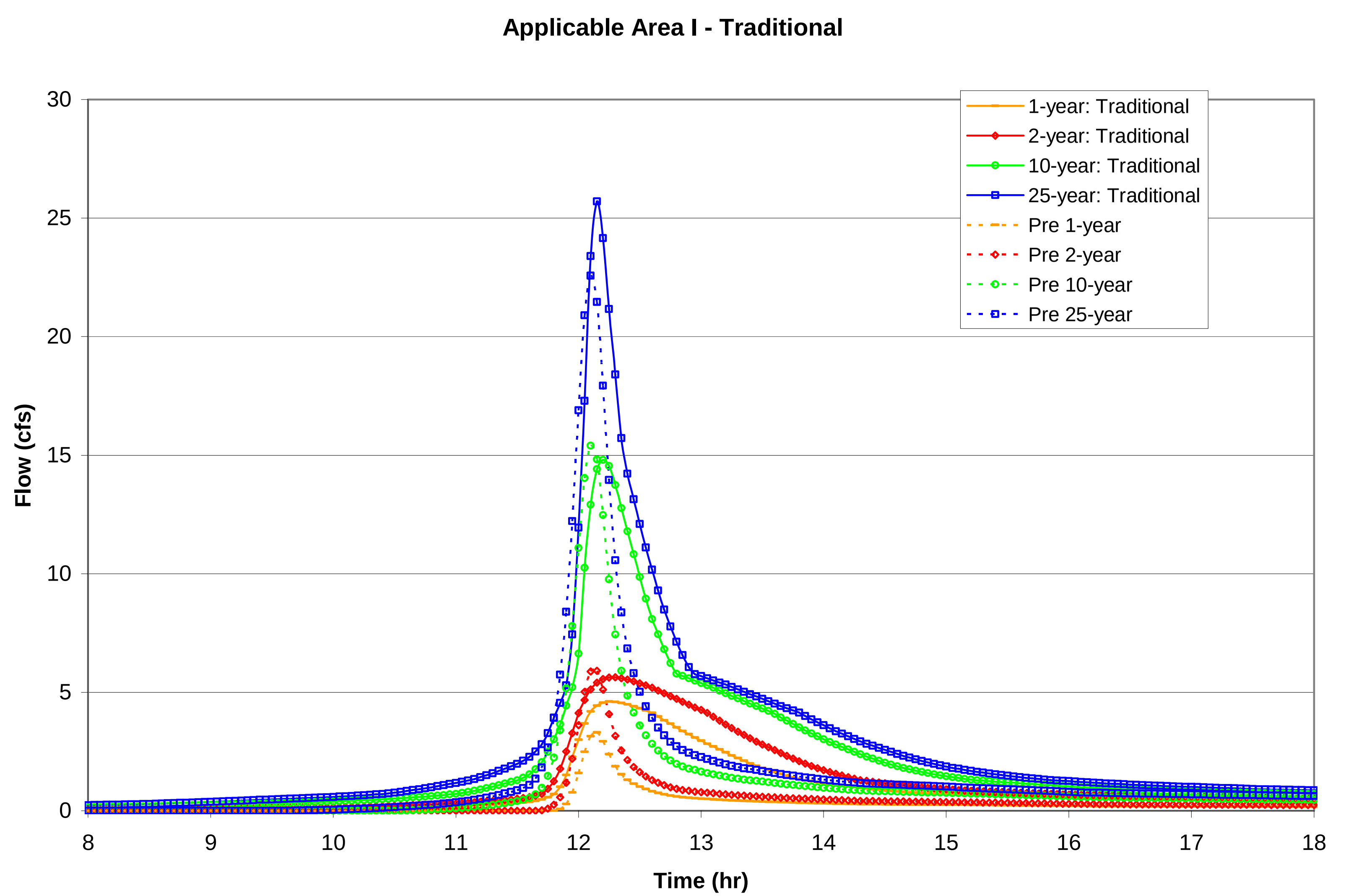
J-7. Pre- and post-development hydrographs for applicable area F with the traditional design.



J-8. Pre- and post-development hydrographs for applicable area G with the traditional design.



J-9. Pre- and post-development hydrographs for applicable area H with the traditional design.



J-10. Pre- and post-development hydrographs for applicable area I with the traditional design.

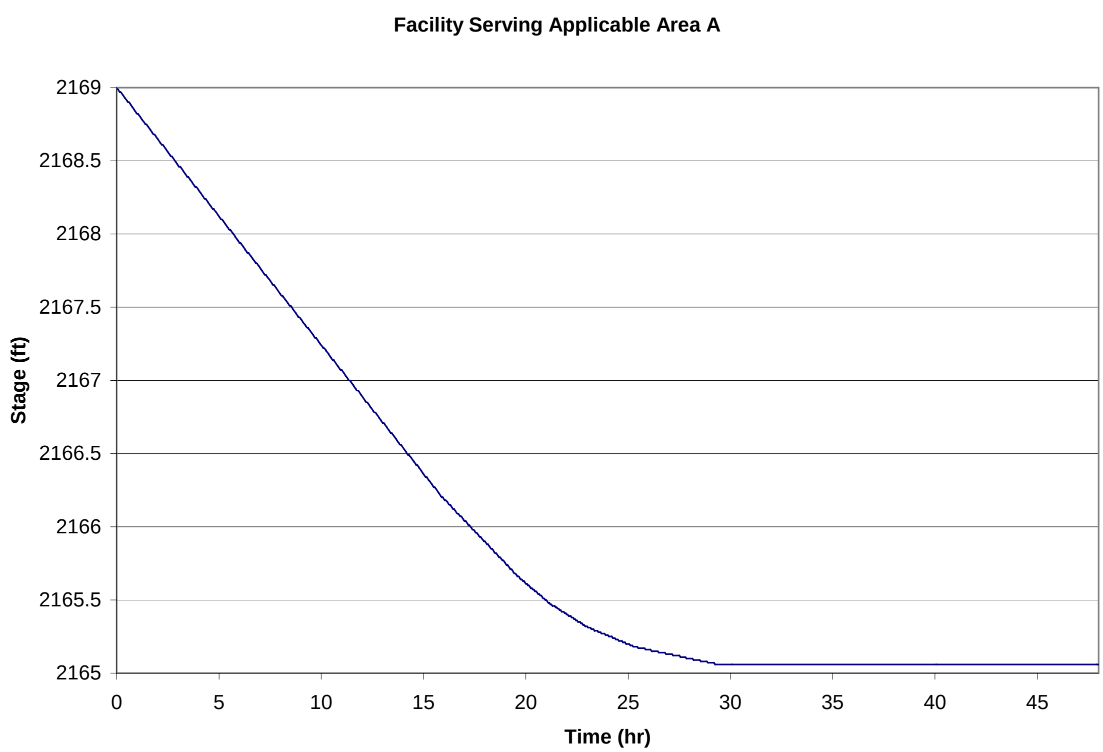
Appendix K: Information for Traditional with WQ component strategy scenario

- **K-1: Water quality volume calculation with corresponding stage in facility serving applicable area.**
- **K-2: 30-hour drawdown for the water quality volume for facility serving Applicable Area A.**
- **K-3: 30-hour drawdown for the water quality volume for facility serving Applicable Area B.**
- **K-4: 30-hour drawdown for the water quality volume for facility serving Applicable Area C.**
- **K-5: 30-hour drawdown for the water quality volume for facility serving Applicable Area D.**
- **K-6: 30-hour drawdown for the water quality volume for facility serving Applicable Area E.**
- **K-7: 30-hour drawdown for the water quality volume for facility serving Applicable Area F.**
- **K-8: 30-hour drawdown for the water quality volume for facility serving Applicable Area G.**
- **K-9: 30-hour drawdown for the water quality volume for facility serving Applicable Area H.**
- **K-10: 30-hour drawdown for the water quality volume for facility serving Applicable Area I.**
- **K-11: Control structure design for the Traditional with Water Quality strategy applied to each applicable area.**
- **K-12: Summary of pre- and post-development peak flows based on the above design for the Traditional with Water Quality component strategy.**
- **K-13: Pre- and post-development hydrographs for applicable area A with the traditional with water quality design.**
- **K-14: Pre- and post-development hydrographs for applicable area B with the traditional with water quality design.**
- **K-15: Pre- and post-development hydrographs for applicable area C with the traditional with water quality design.**

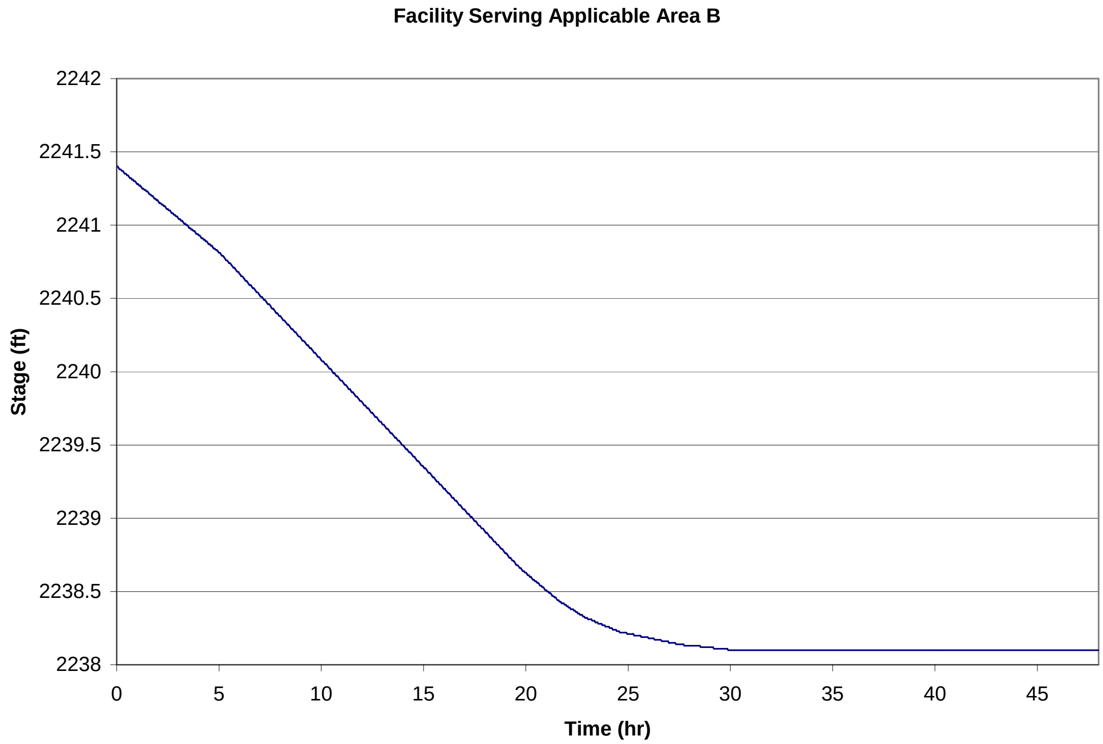
- **K-16: Pre- and post-development hydrographs for applicable area D with the traditional with water quality design.**
- **K-17: Pre- and post-development hydrographs for applicable area E with the traditional with water quality design.**
- **K-18: Pre- and post-development hydrographs for applicable area F with the traditional with water quality design.**
- **K-19: Pre- and post-development hydrographs for applicable area G with the traditional with water quality design.**
- **K-20: Pre- and post-development hydrographs for applicable area H with the traditional with water quality design.**
- **K-21: Pre- and post-development hydrographs for applicable area I with the traditional with water quality design.**

K-1. Water quality volume calculation with corresponding stage in facility serving applicable area. Starting water surface elevation is set at the WQ volume stage for sizing of extended detention orifice.

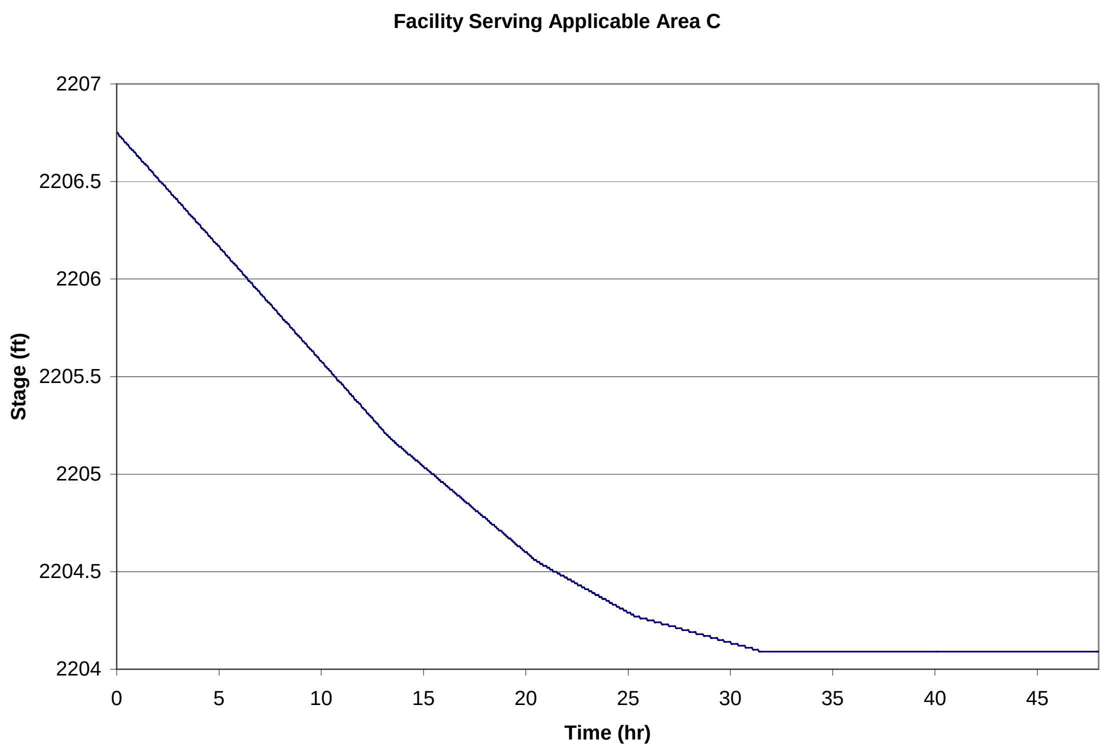
Applicable Area	Impervious Area		WQ Volume (ft^3)	Corresponding stage in serving facility (ft)
	(acres)	(ft^2)		
A	10.57	460,590	38,383	2169.00
B	2.38	103,574	8,631	2241.40
C	3.48	151,636	12,636	2206.75
D	1.30	56,466	4,705	2161.82
E	3.34	145,451	12,121	2138.76
F	1.19	51,688	4,307	2123.90
G	1.38	60,049	5,004	2112.26
H	1.52	66,064	5,505	2099.05
I	3.43	149,525	12,460	2080.16



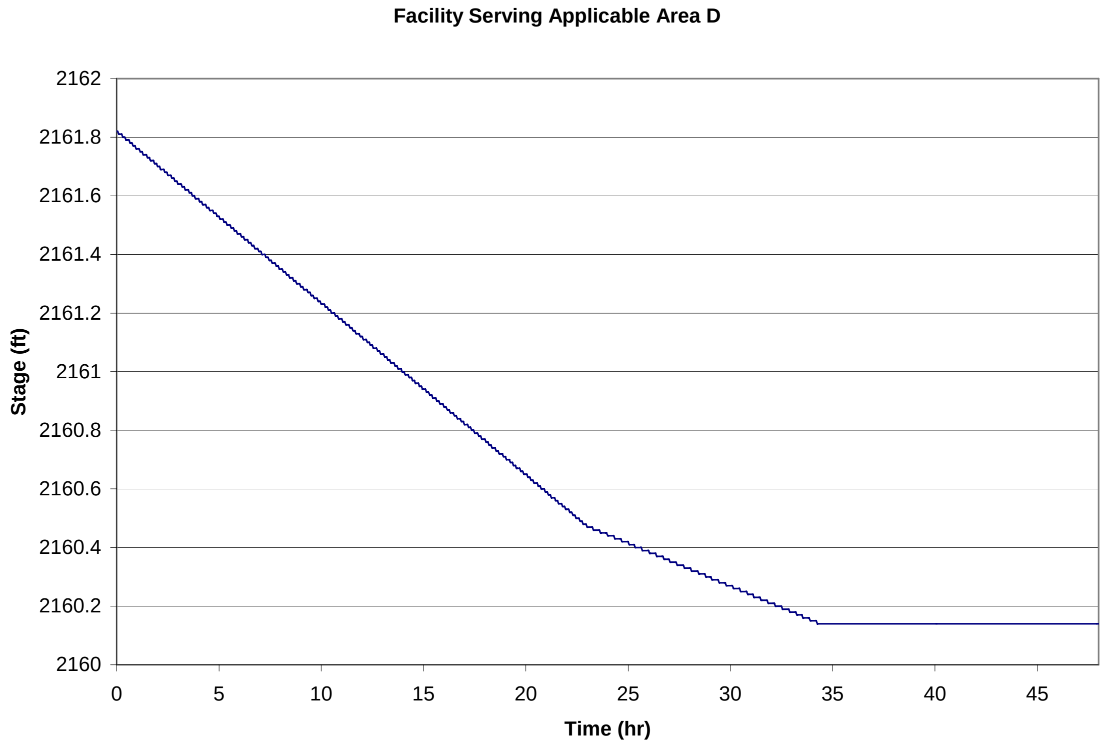
K-2. 30-hour drawdown for the water quality volume for facility serving Applicable Area A. Drawdown is achieved with a 3.5" orifice set at the invert of the facility.



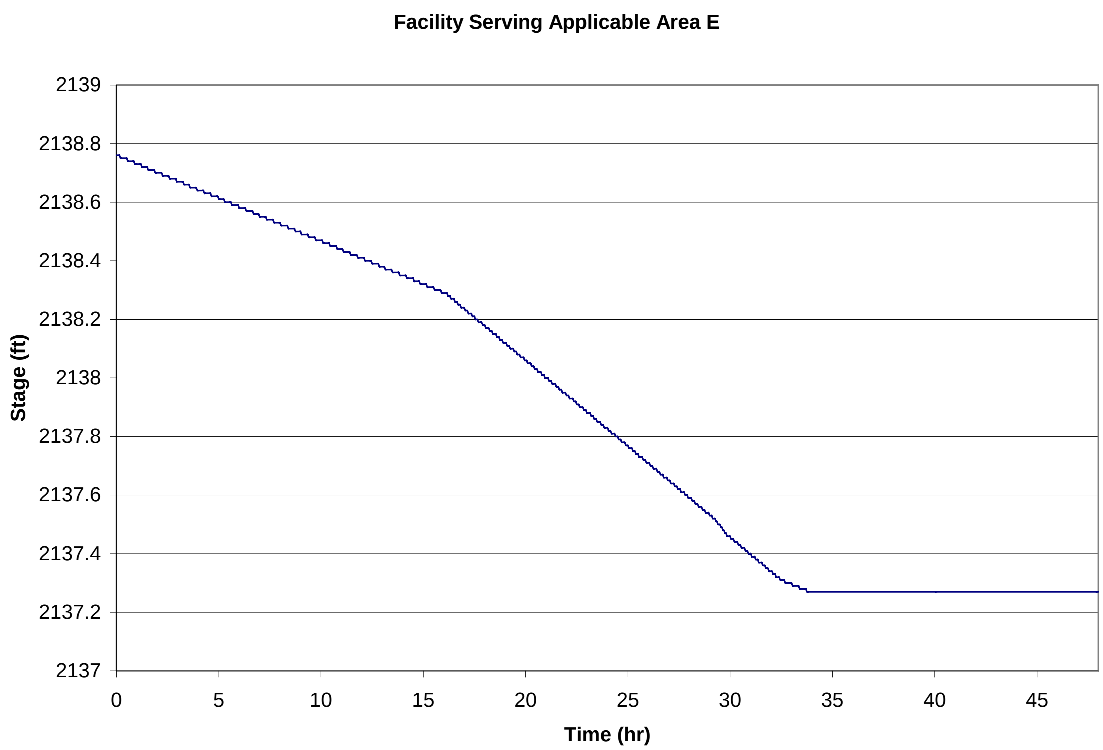
K-3. 30-hour drawdown for the water quality volume for facility serving Applicable Area B. Drawdown is achieved with a 1.75" orifice set at the invert of the facility.



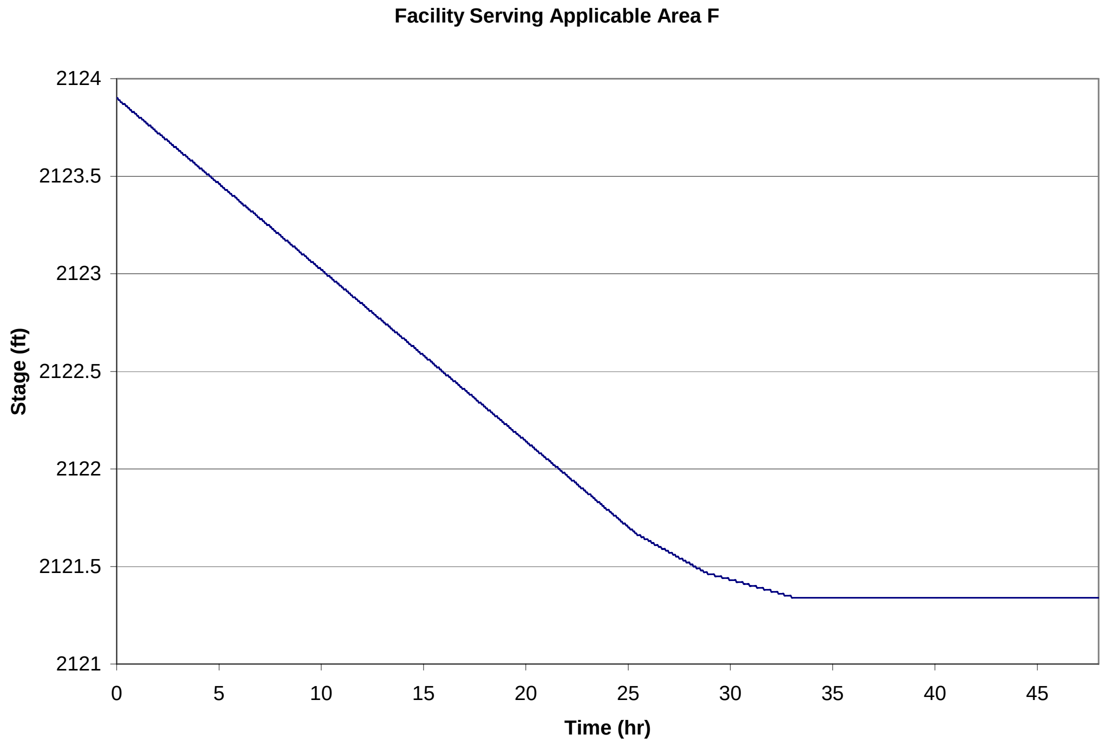
K-4. 30-hour drawdown for the water quality volume for facility serving Applicable Area C. Drawdown is achieved with a 2.10" orifice set at the invert of the facility.



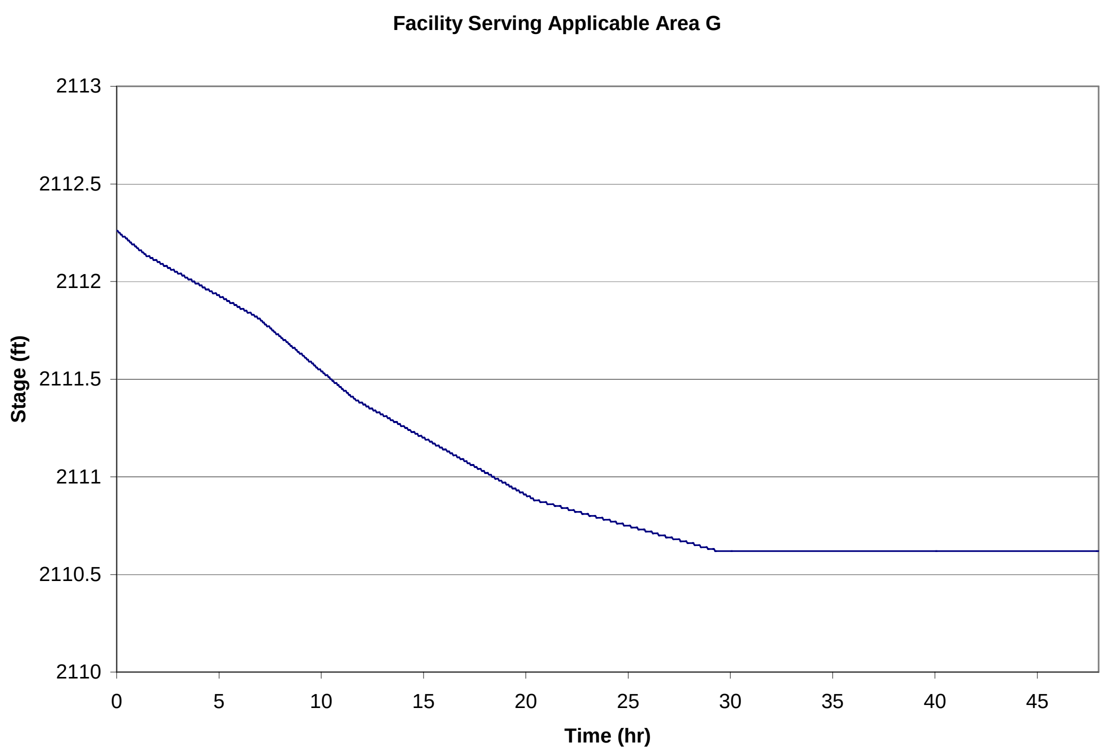
K-5. 30-hour drawdown for the water quality volume for facility serving Applicable Area D. Drawdown is achieved with a 1.3" orifice set at the invert of the facility.



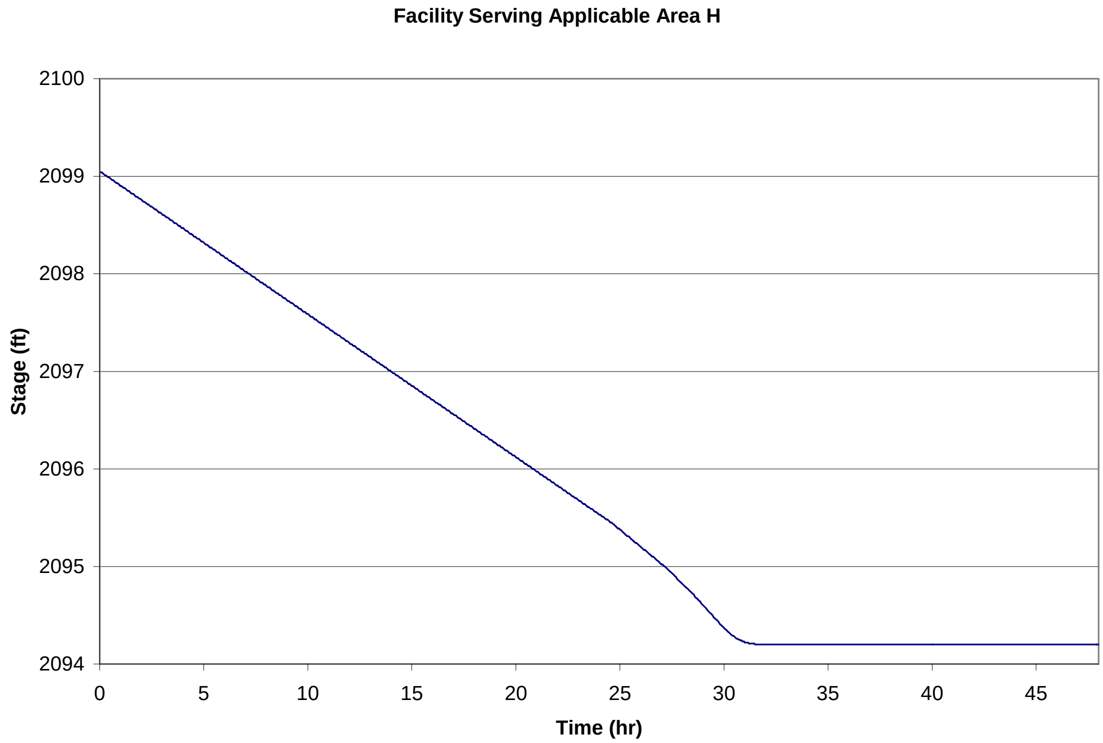
K-6. 30-hour drawdown for the water quality volume for facility serving Applicable Area E. Drawdown is achieved with 2-0.5" orifices set at the invert of the facility.



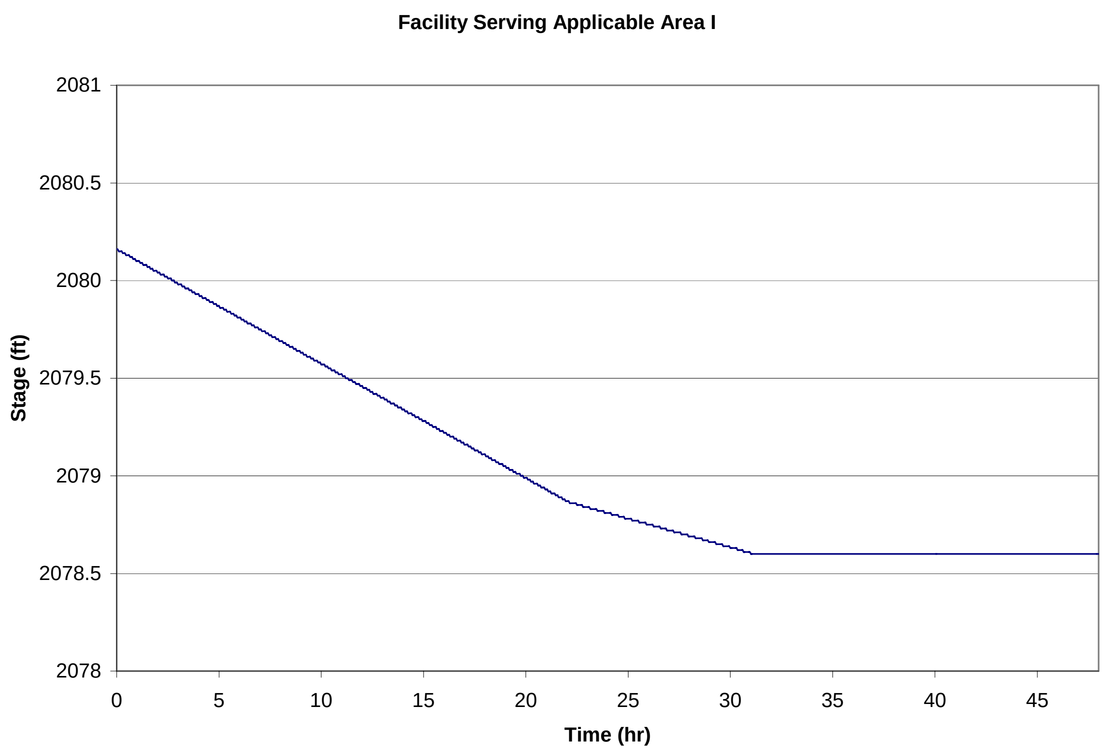
K-7. 30-hour drawdown for the water quality volume for facility serving Applicable Area F. Drawdown is achieved with 1.2" orifice set at the invert of the facility.



K-8. 30-hour drawdown for the water quality volume for facility serving Applicable Area G. Drawdown is achieved with 1.5" orifice set at the invert of the facility.



K-9. 30-hour drawdown for the water quality volume for facility serving Applicable Area H. Drawdown is achieved with 1.1" orifice set at the invert of the facility.



K-10. 30-hour drawdown for the water quality volume for facility serving Applicable Area I. Drawdown is achieved with 2.5" orifice set at the invert of the facility.

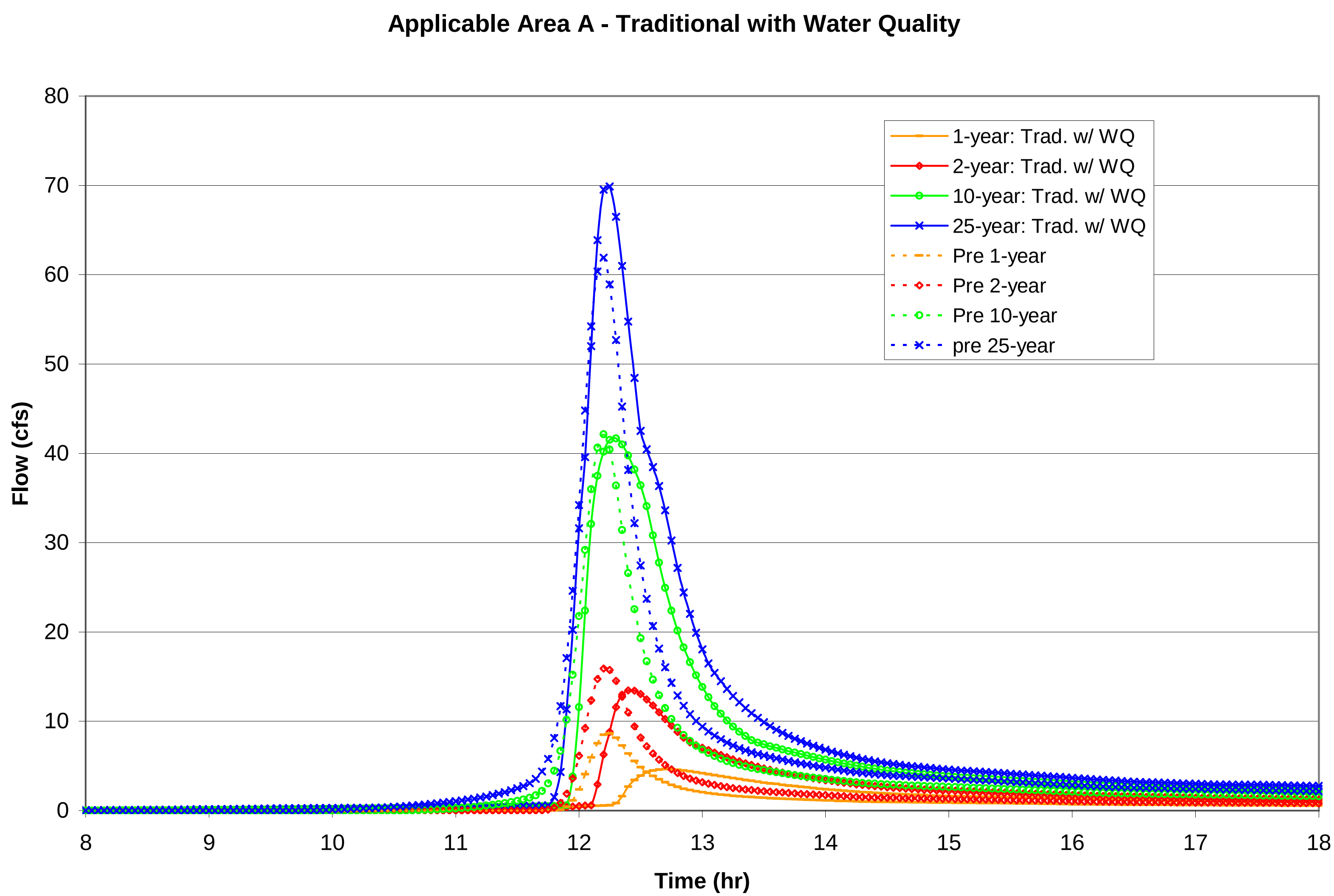
K-11. Control structure design for the Traditional with Water Quality strategy applied to each applicable area. Resulting hydrographs are provided below as part of this Appendix.

Applicable Area	WQ Orifice size		2-year Orifice		10-year Riser or orifice		25-year Weir or Riser	
	Dia (in)	Invert (ft)	Size	Invert (ft)	Size	Crest (ft)	Size	Crest (ft)
A	3.5	2,165.00	24" Dia.	2,169.00	N/A		48" r	2,171.90
B	1.8	2,238.10	18" Dia.	2,241.40	N/A		30' w	2,243.70
C	2.1	2,204.00	20'W x 6'H	2,206.75	30" r	2,207.85	15' w	2209.4
D	1.3	2,160.00	13" Dia	2,161.82	18" r	2,163.20	10' w	2,164.60
E	0.5	2,137.26	15" Dia	2,138.80	23" o	2,143.00	25' w	2,144.80
F	1.2	2,121.30	12" Dia	2,123.90	7.5" r	2,125.10	30' w	2,126.50
G	1.5	2,110.50	12" Dia	2,112.26	3.5" r	2,113.00	15' w	2,113.80
H	1.1	2,094.20	7" Dia	2,099.05	8" r	2,100.40	15' w	2,101.70
I	2.5	2,078.50	21" Dia	2,080.16	48 r	2,082.40	48" r	2,082.40

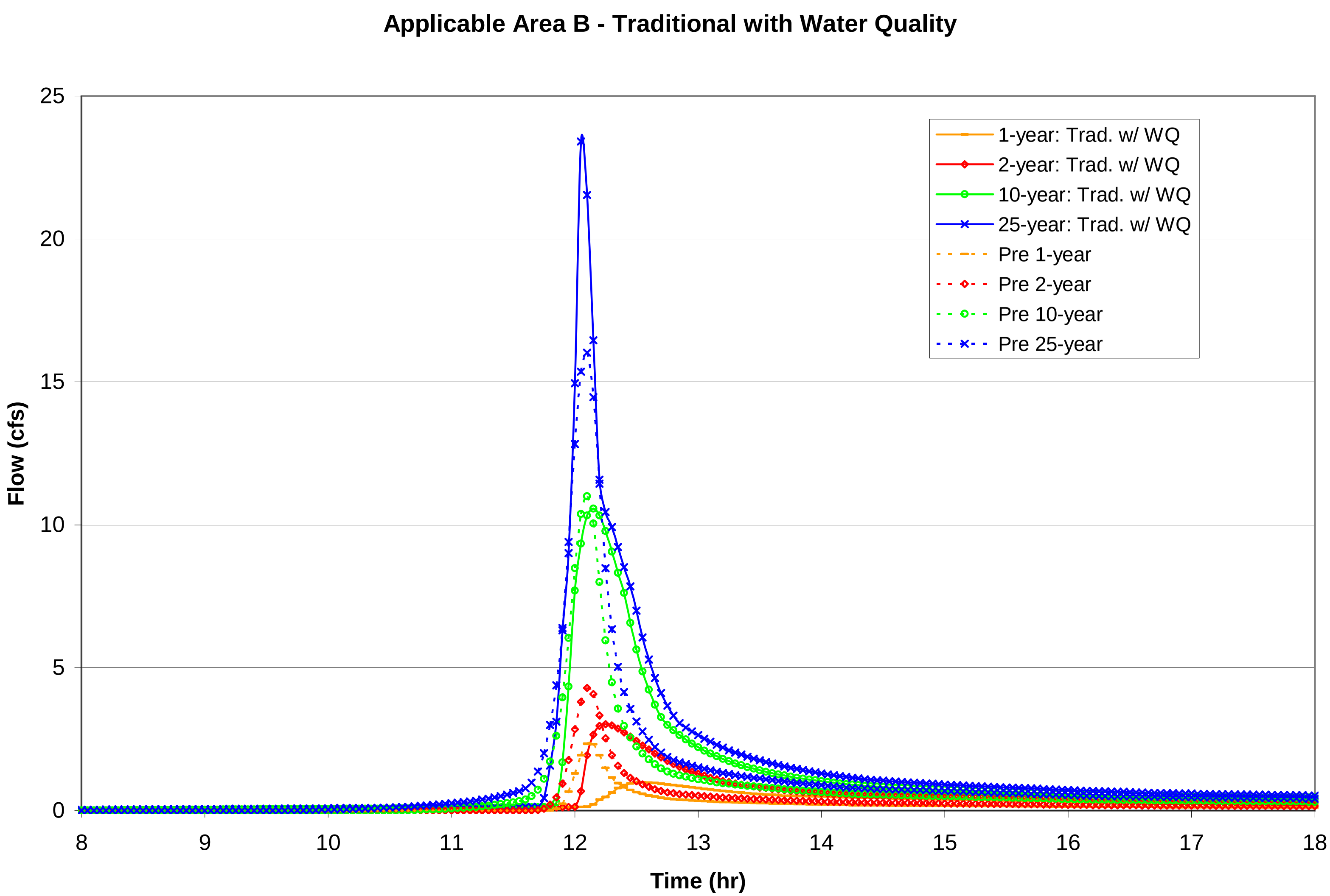
* An 'r' indicates where a riser is used; an 'o' indicates an orifice; a 'w' indicates a rectangular weir.

K-12. Summary of pre- and post-development peak flows based on the above design for the Traditional with Water Quality component strategy.

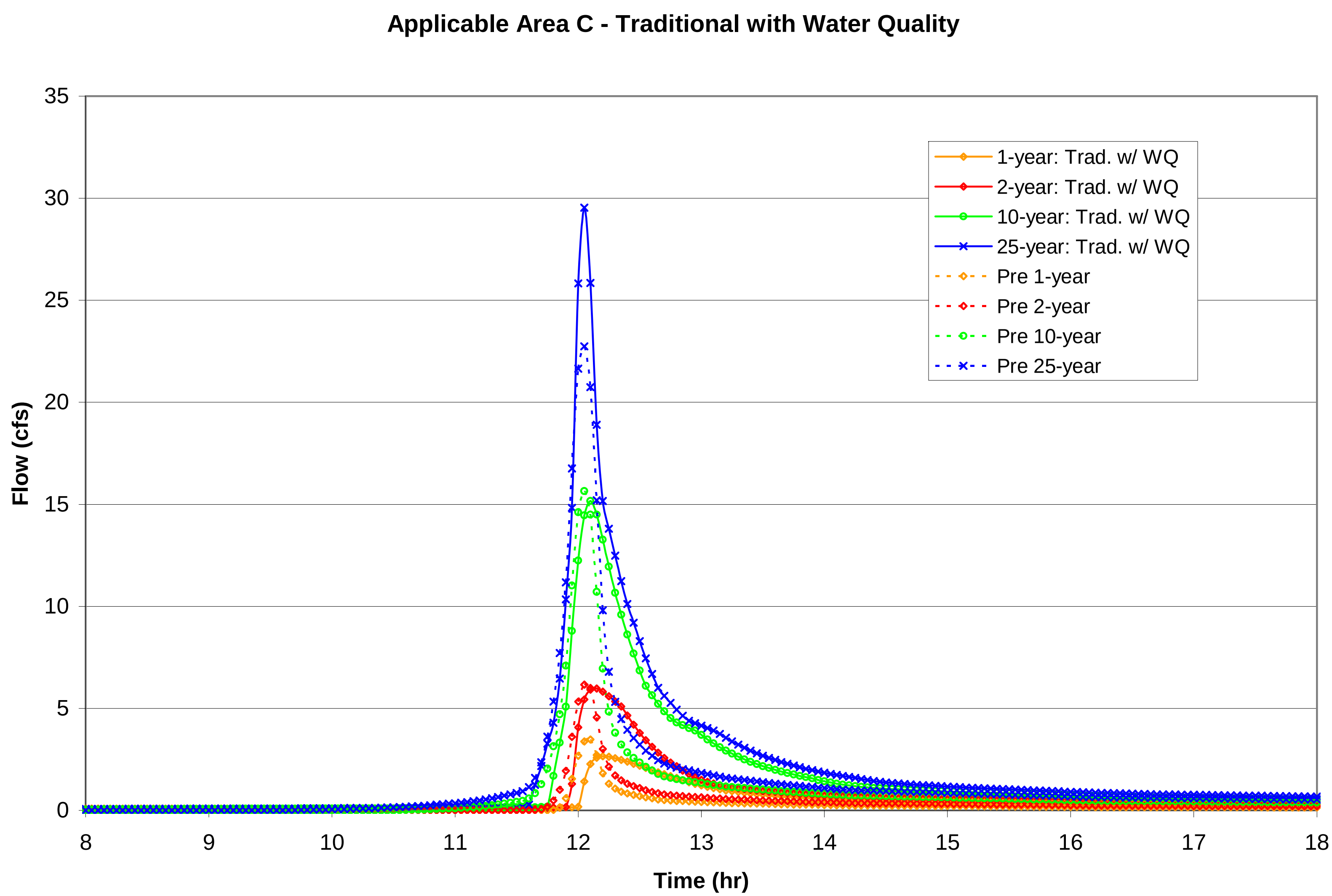
Applicable Area	2-year		10-year	
	pre peak (cfs)	post-peak (cfs)	pre peak (cfs)	post-peak (cfs)
A	15.92	13.47	42.13	41.68
B	4.29	3.03	11	10.57
C	6.16	5.97	15.65	14.43
D	4.29	4.00	11.06	10.83
E	12.28	11.08	32.25	31.96
F	3.03	3.03	7.61	7.53
G	1.59	1.45	4.15	4.15
H	1.57	1.33	3.95	3.83
I	5.91	5.18	15.4	13.5



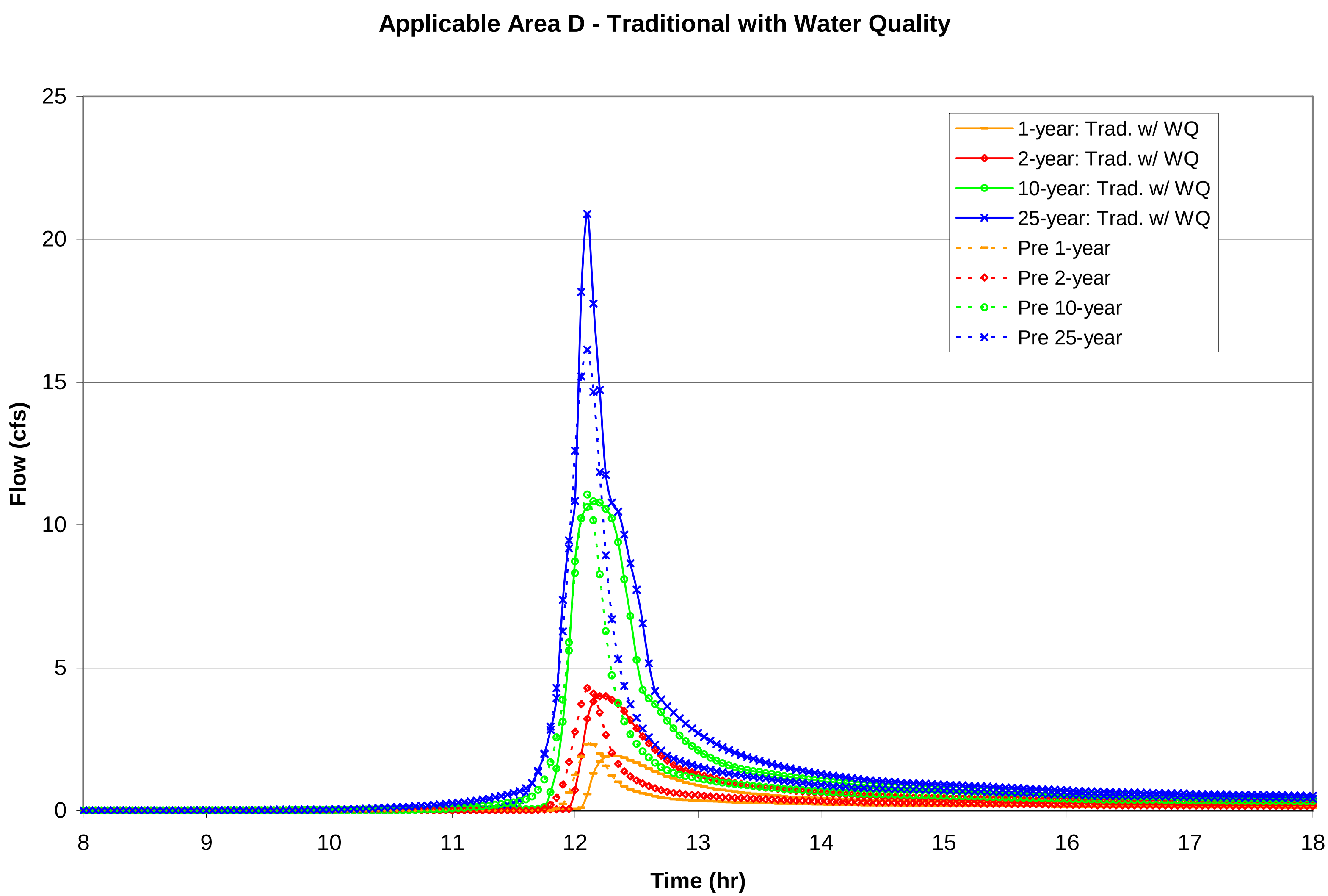
K-13. Pre- and post-development hydrographs for applicable area A with the traditional with water quality design.



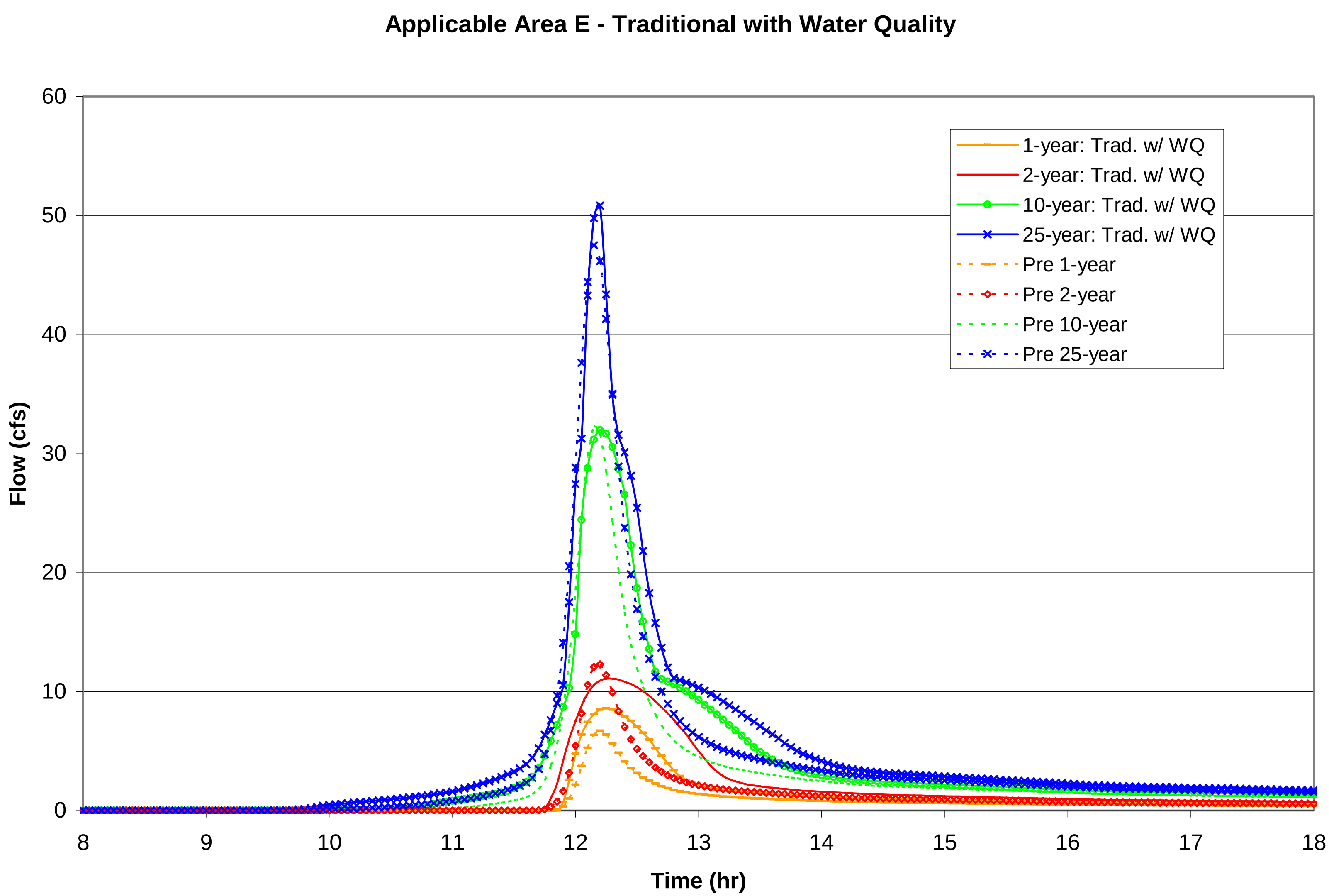
K-14. Pre- and post-development hydrographs for applicable area B with the traditional with water quality design.



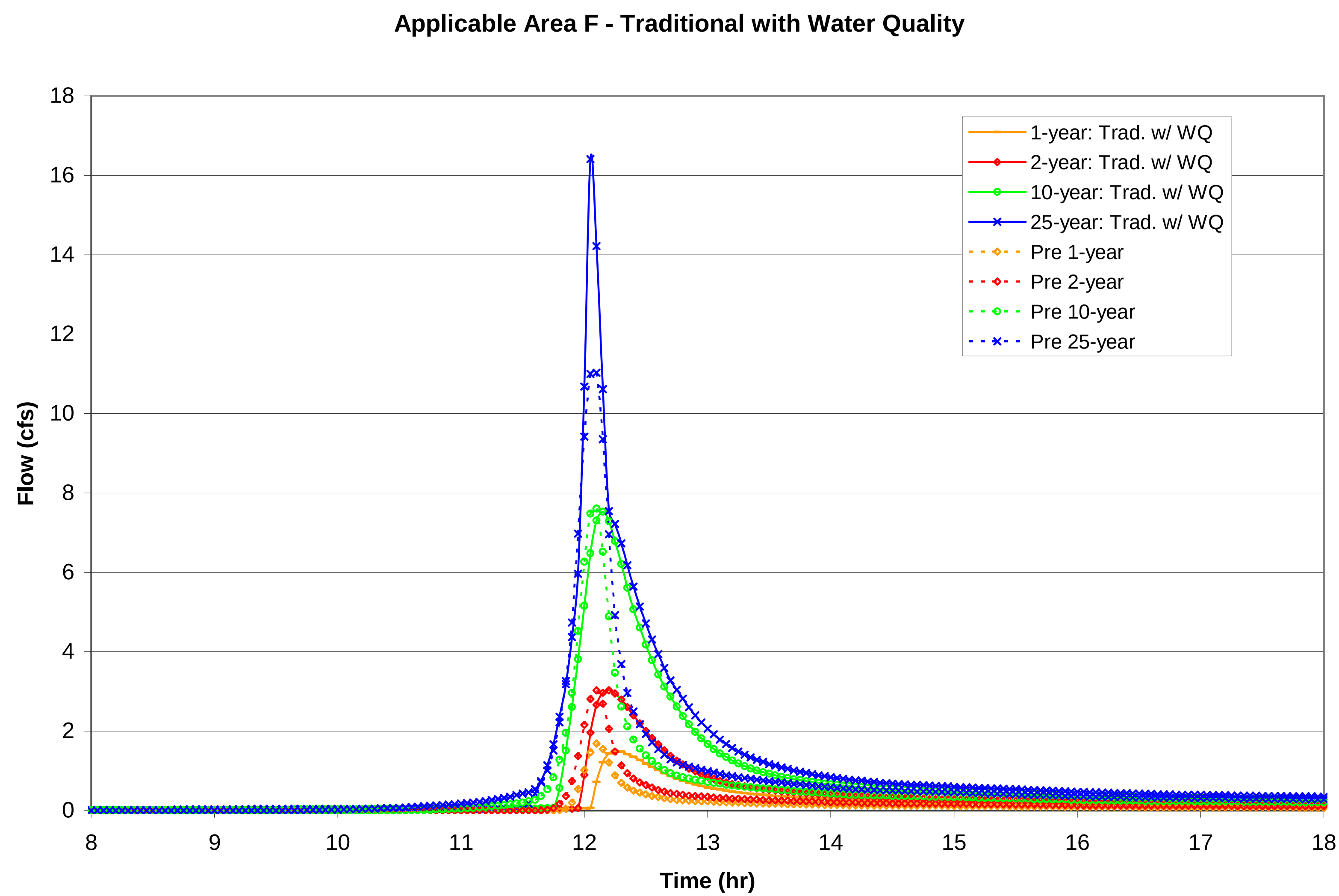
K-15. Pre- and post-development hydrographs for applicable area C with the traditional with water quality design.



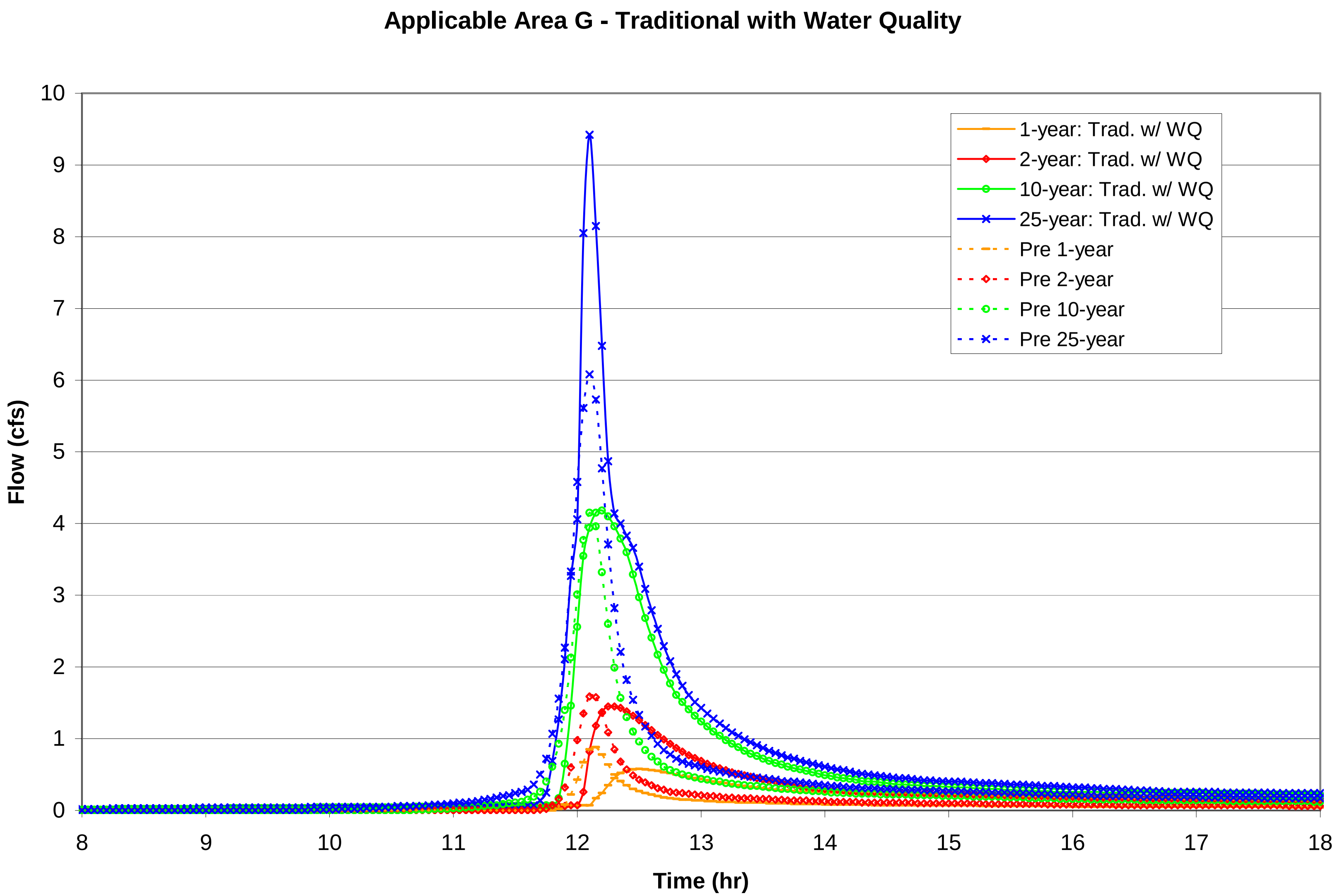
K-16. Pre- and post-development hydrographs for applicable area D with the traditional with water quality design.



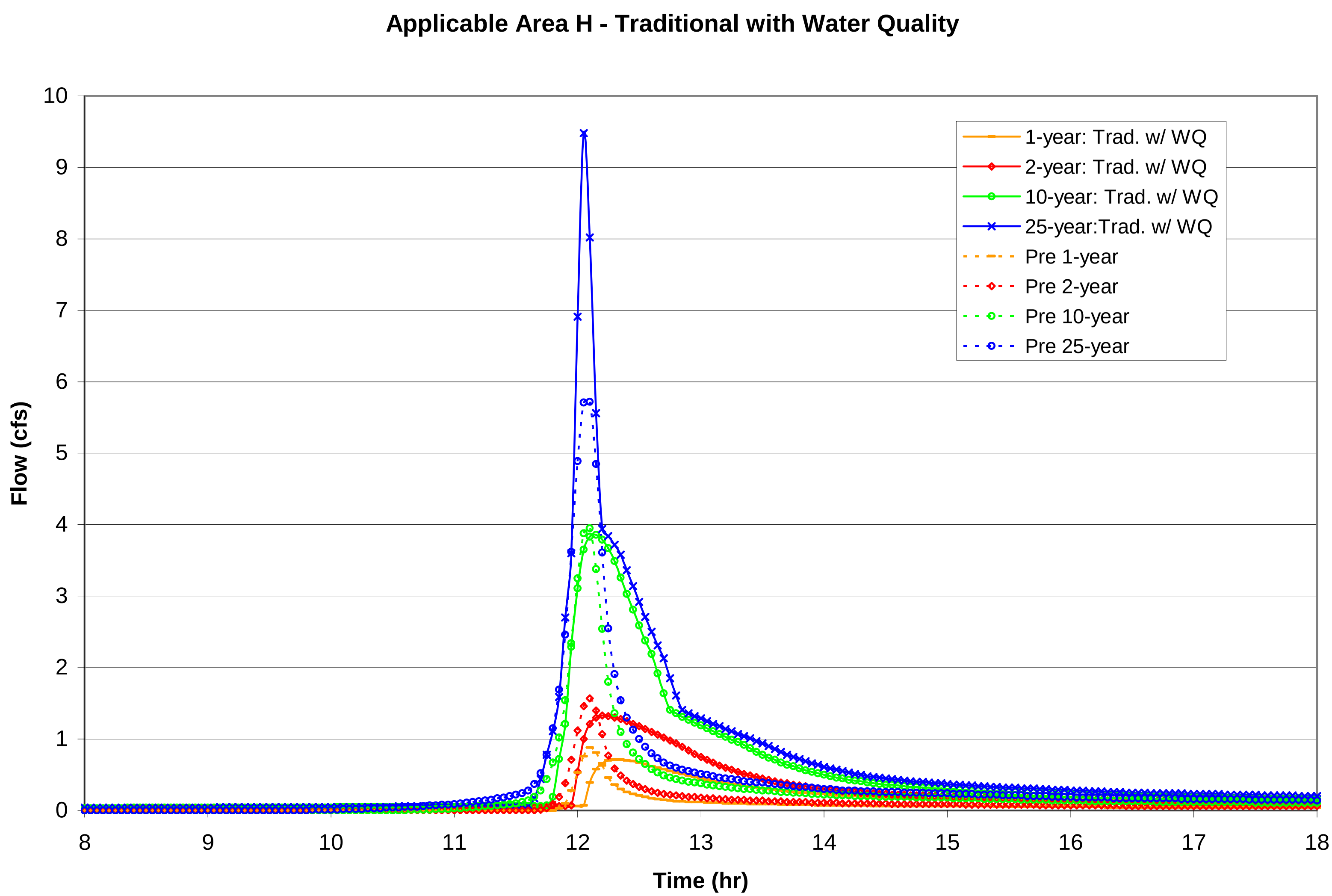
K-17. Pre- and post-development hydrographs for applicable area E with the traditional with water quality design.



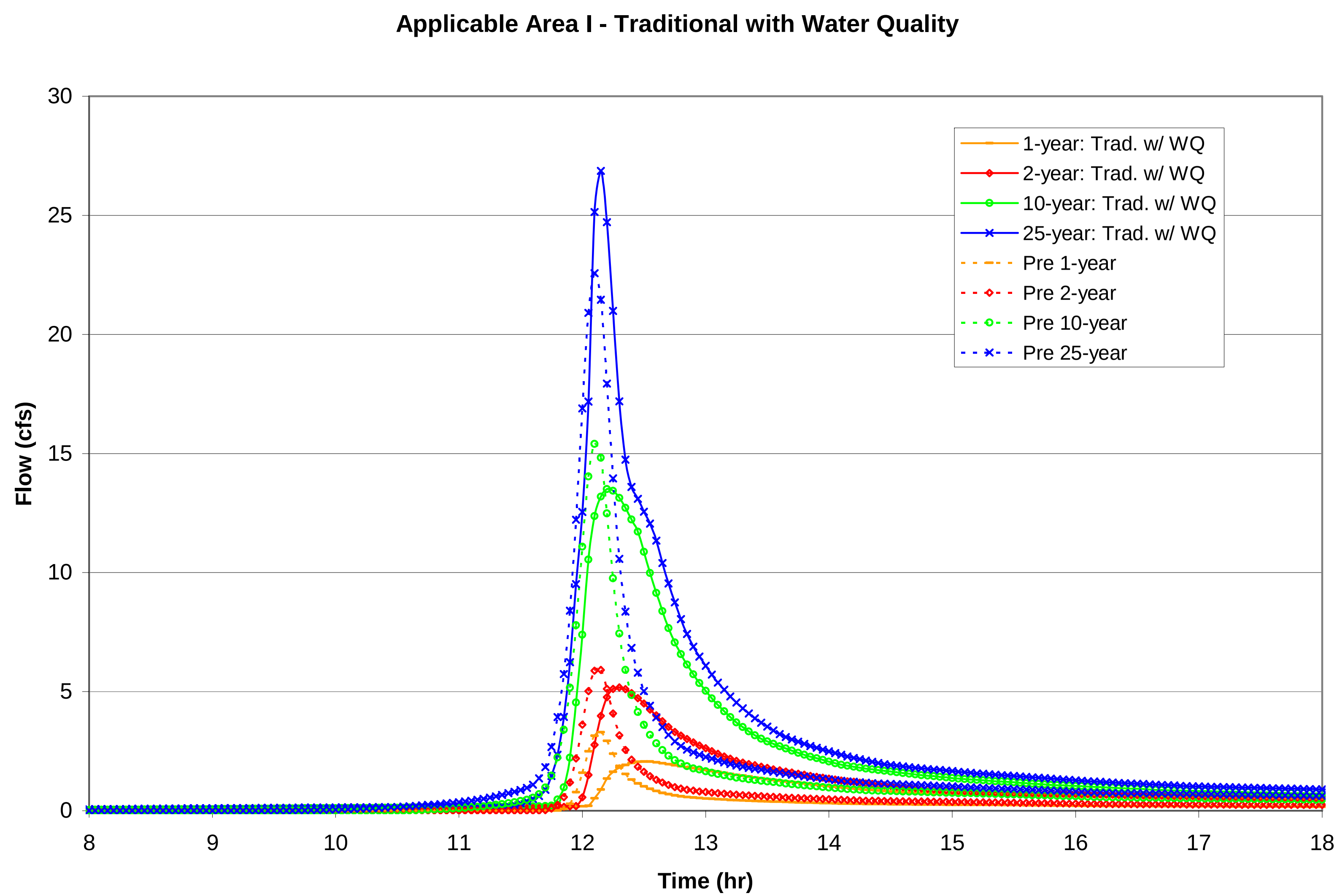
K-18. Pre- and post-development hydrographs for applicable area F with the traditional with water quality design.



K-19. Pre- and post-development hydrographs for applicable area G with the traditional with water quality design.



K-20. Pre- and post-development hydrographs for applicable area H with the traditional with water quality design.



K-21. Pre- and post-development hydrographs for applicable area I with the traditional with water quality design.

Appendix L: Information for the Unit Release Rate strategy

- **L-1: Information for application of the determined unit release rates for the 2-year design storm.**
- **L-2: Information for application of the determined unit release rates for the 25-year design storm.**
- **L-3: Control structure design information for achieving the URR strategy allowable release rates.**
- **L-4: Post-development hydrographs for applicable area A with the Unit Release Rate design. Predevelopment hydrographs are provided for comparison.**
- **L-5: Post-development hydrographs for applicable area B with the Unit Release Rate design. Predevelopment hydrographs are provided for comparison.**
- **L-6: Post-development hydrographs for applicable area C with the Unit Release Rate design. Predevelopment hydrographs are provided for comparison.**
- **L-7: Post-development hydrographs for applicable area D with the Unit Release Rate design. Predevelopment hydrographs are provided for comparison.**
- **L-8: Post-development hydrographs for applicable area E with the Unit Release Rate design. Predevelopment hydrographs are provided for comparison.**
- **L-9: Post-development hydrographs for applicable area F with the Unit Release Rate design. Predevelopment hydrographs are provided for comparison.**
- **L-10: Post-development hydrographs for applicable area G with the Unit Release Rate design. Predevelopment hydrographs are provided for comparison.**
- **L-11: Post-development hydrographs for applicable area H with the Unit Release Rate design. Predevelopment hydrographs are provided for comparison.**

- **L12: Post-development hydrographs for applicable area I with the Unit Release Rate design. Predevelopment hydrographs are provided for comparison.**

L-1. Information for application of the determined unit release rates for the 2-year design storm. *Area* reflects area of applicable area; *allowable release rate* is the area multiplied by the URR and *design* value is the actual release rate after modification of the control structure.

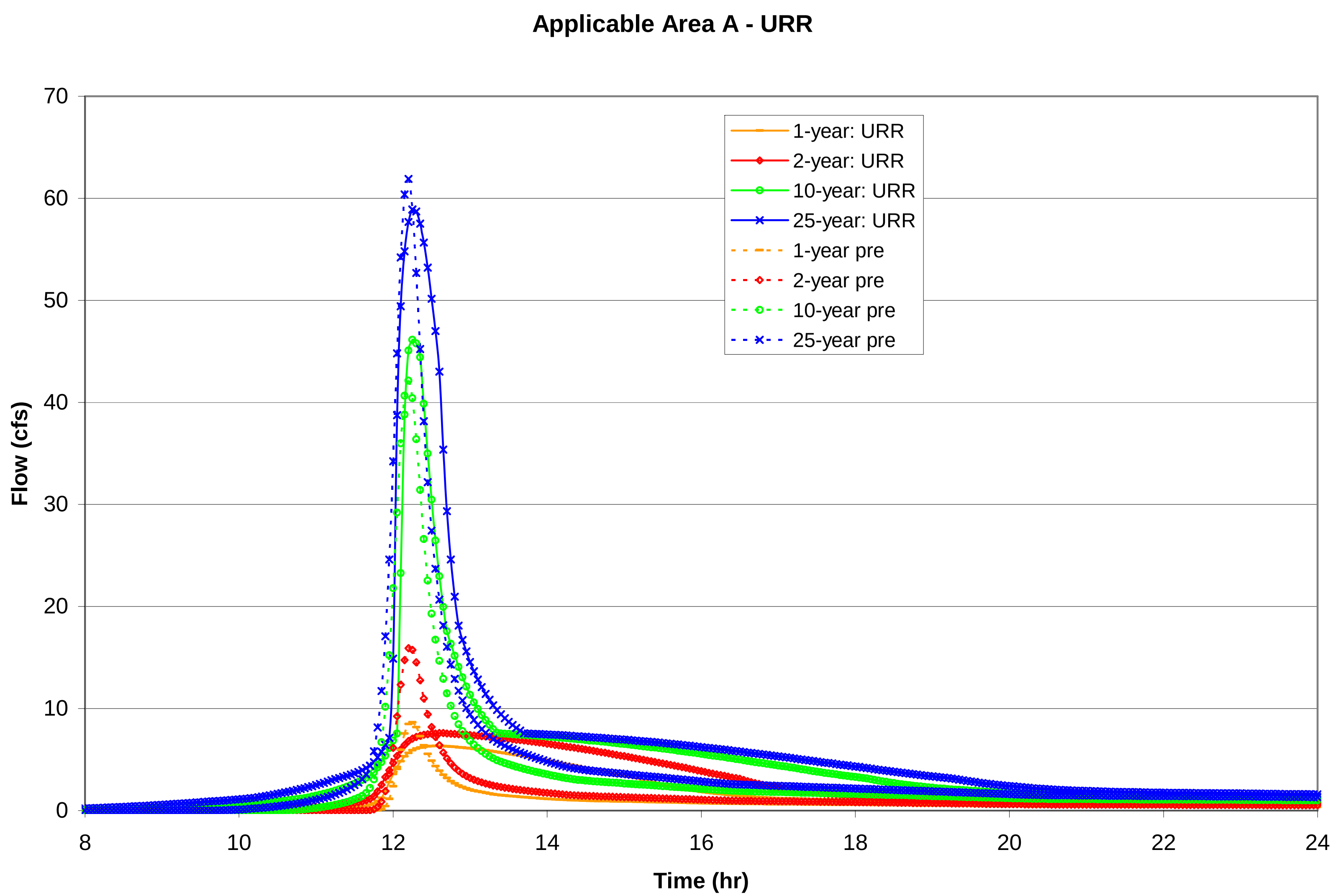
Site ID	Area (ac)	2-yr URR (cfs/acre)	2-yr Allowable Release rate (cfs)	Design (cfs)
A	30.27	0.25	7.57	7.54
B	5.97	0.25	1.49	1.31
C	7.56	0.25	1.89	1.89
D	6.05	0.25	1.51	1.51
E	21.53	0.25	5.38	4.80
F	3.98	0.25	1.00	0.83
G	2.35	0.25	0.59	0.59
H	2.07	0.25	0.52	0.49
I	8.77	0.25	2.19	1.92

L-2. Information for application of the determined unit release rates for the 25-year design storm. *Area* reflects area of applicable area; *allowable release rate* is the area multiplied by the URR and *design* value is the actual release rate after modification of the control structure.

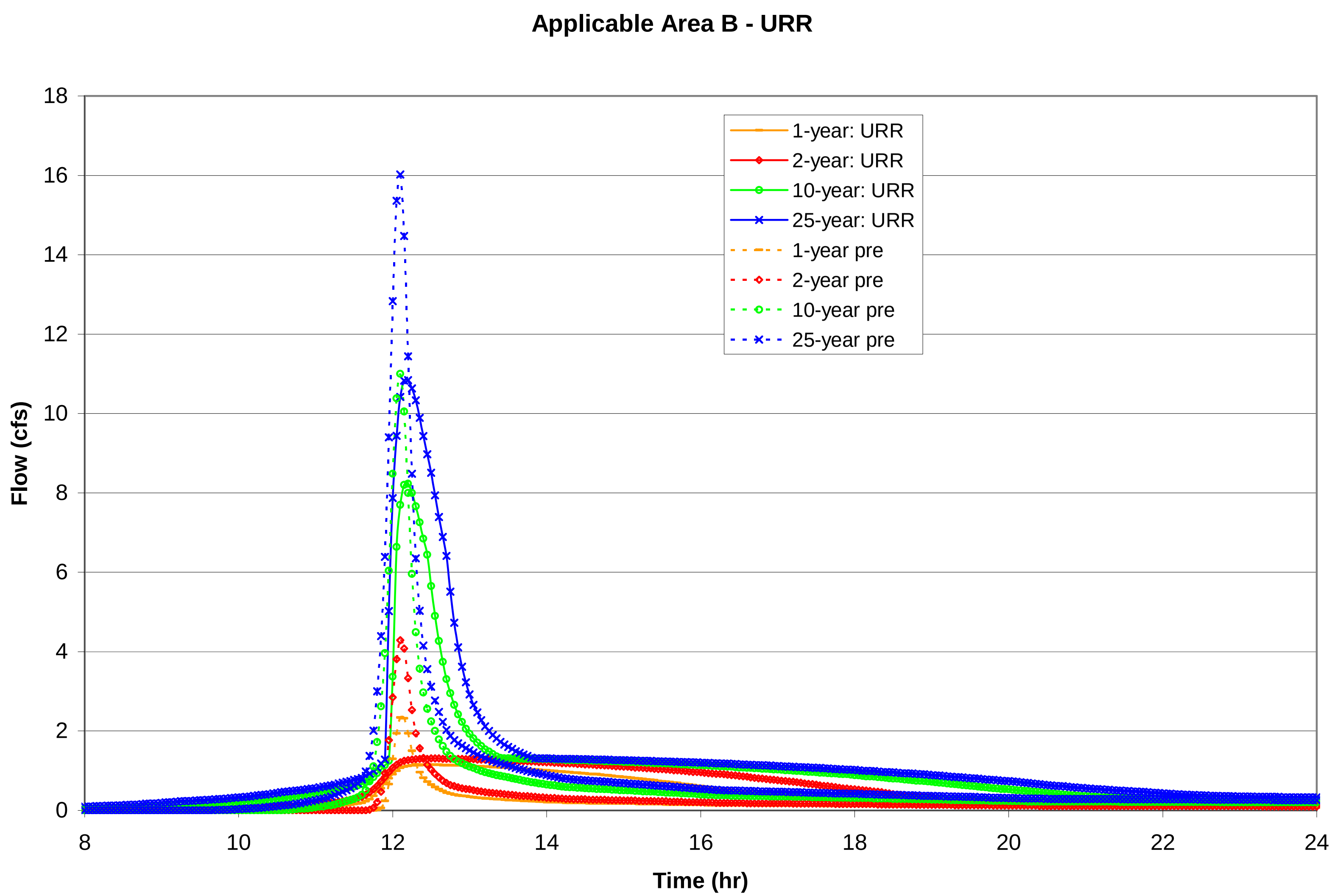
Site ID	Area (ac)	25-yr URR (cfs/acre)	25-yr Allowable Release rate (cfs)	Design (cfs)
A	30.27	2	60.538	58.89
B	5.97	2	11.94	10.85
C	7.56	2	15.12	14.15
D	6.05	2	12.104	10.51
E	21.53	2	43.052	41.23
F	3.98	2	7.96	7.05
G	2.35	2	4.7	4.7
H	2.07	2	4.132	3.76
I	8.77	2	17.546	16.49

L-3. Control structure design information for achieving the URR strategy allowable release rates.

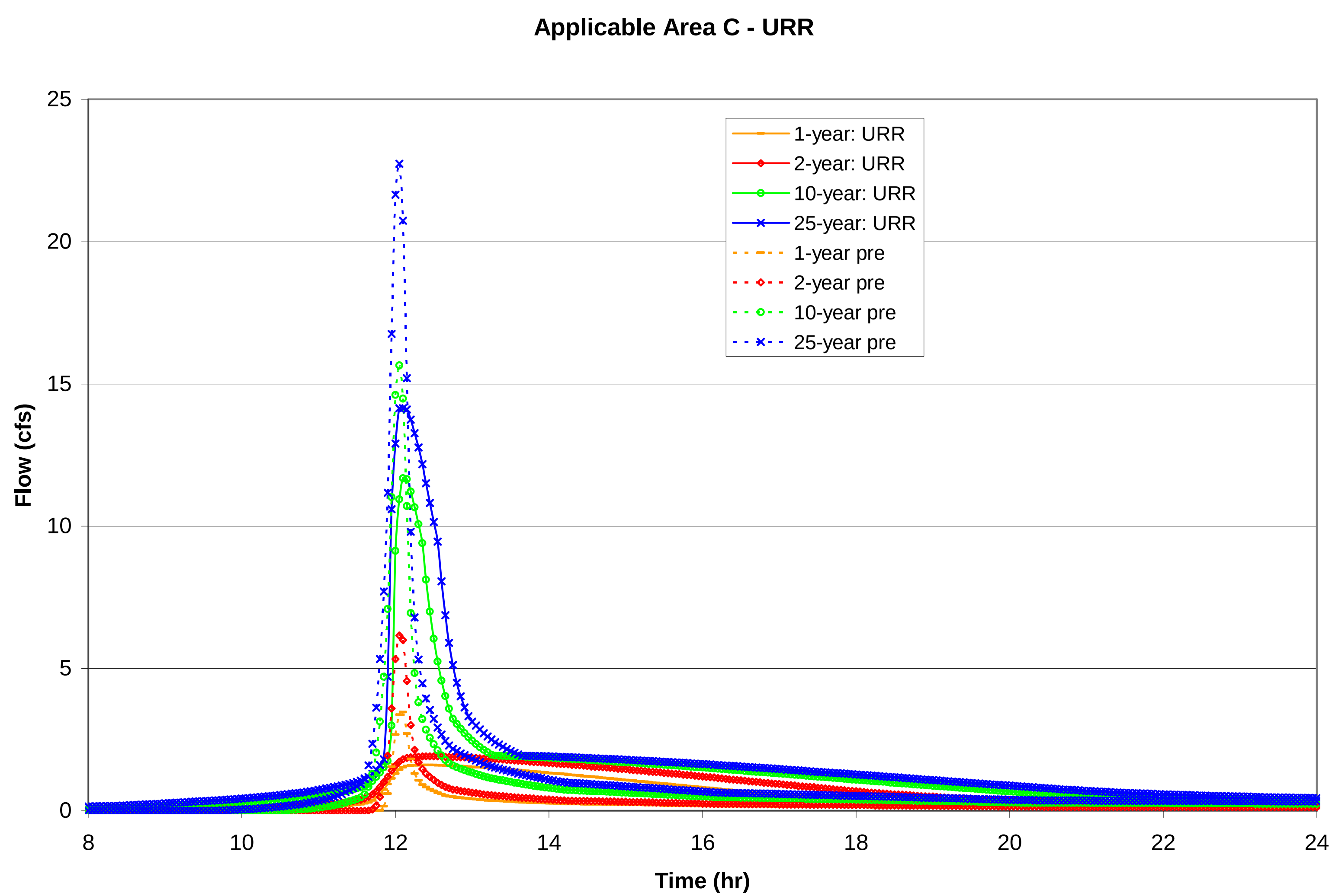
Applicable Area Served	2-year Orifice		25-year Riser	
	Dia. (in)	Invert (ft)	Dia. (in)	Invert (ft)
A	12	2,165.00	36	2,170.10
B	5	2,238.10	15	2,242.30
C	4.5	2,204.00	18	2,208.00
D	6	2,160.00	15	2,163.50
E	8.5	2,137.26	34	2,144.10
F	4	2,121.30	12	2,125.40
G	2	2,110.50	12	2,113.00
H	2	2,094.20	9	2,100.10
I	7	2,078.50	21	2,081.40



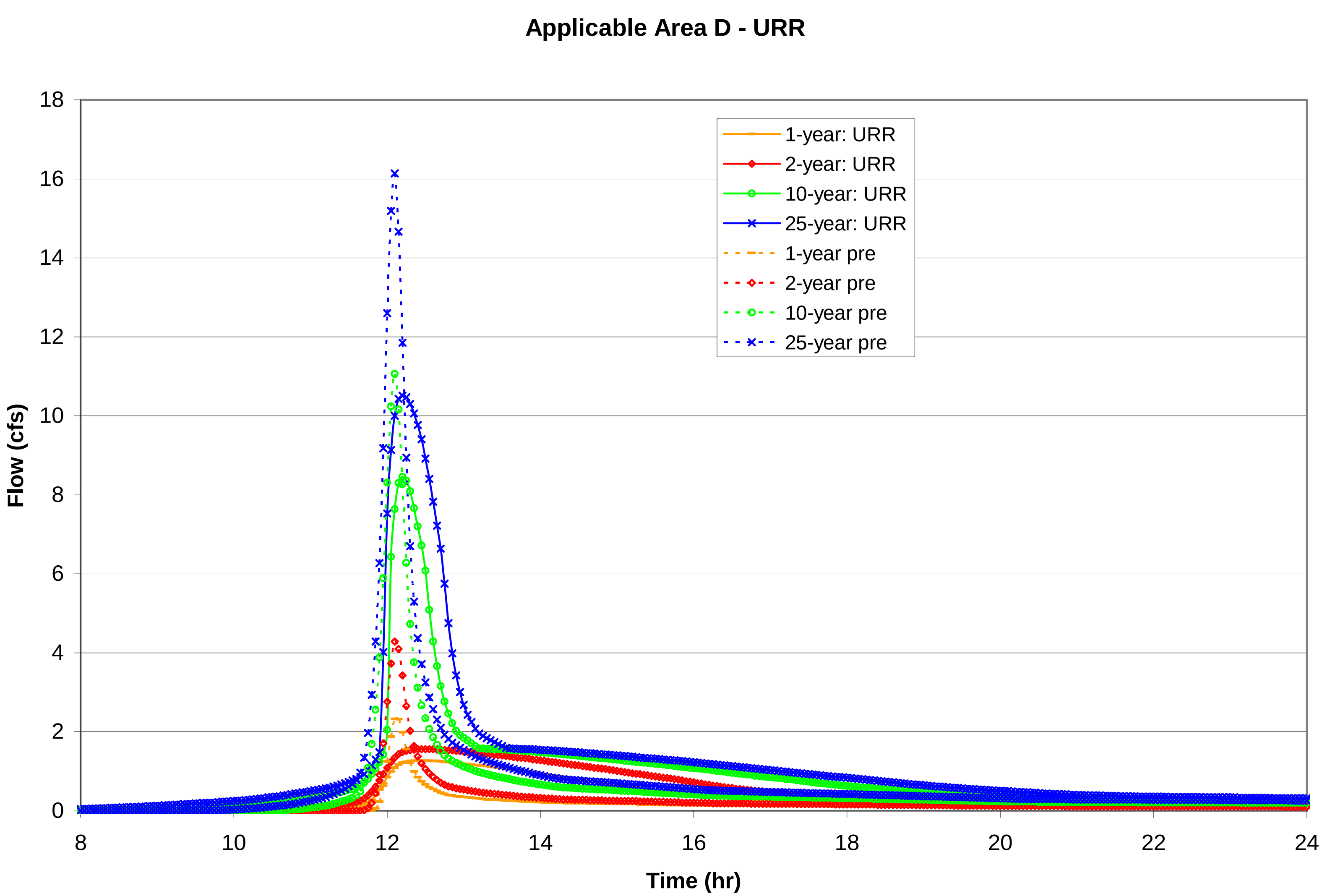
L-4. Post-development hydrographs for applicable area A with the Unit Release Rate design. Predevelopment hydrographs are provided for comparison.



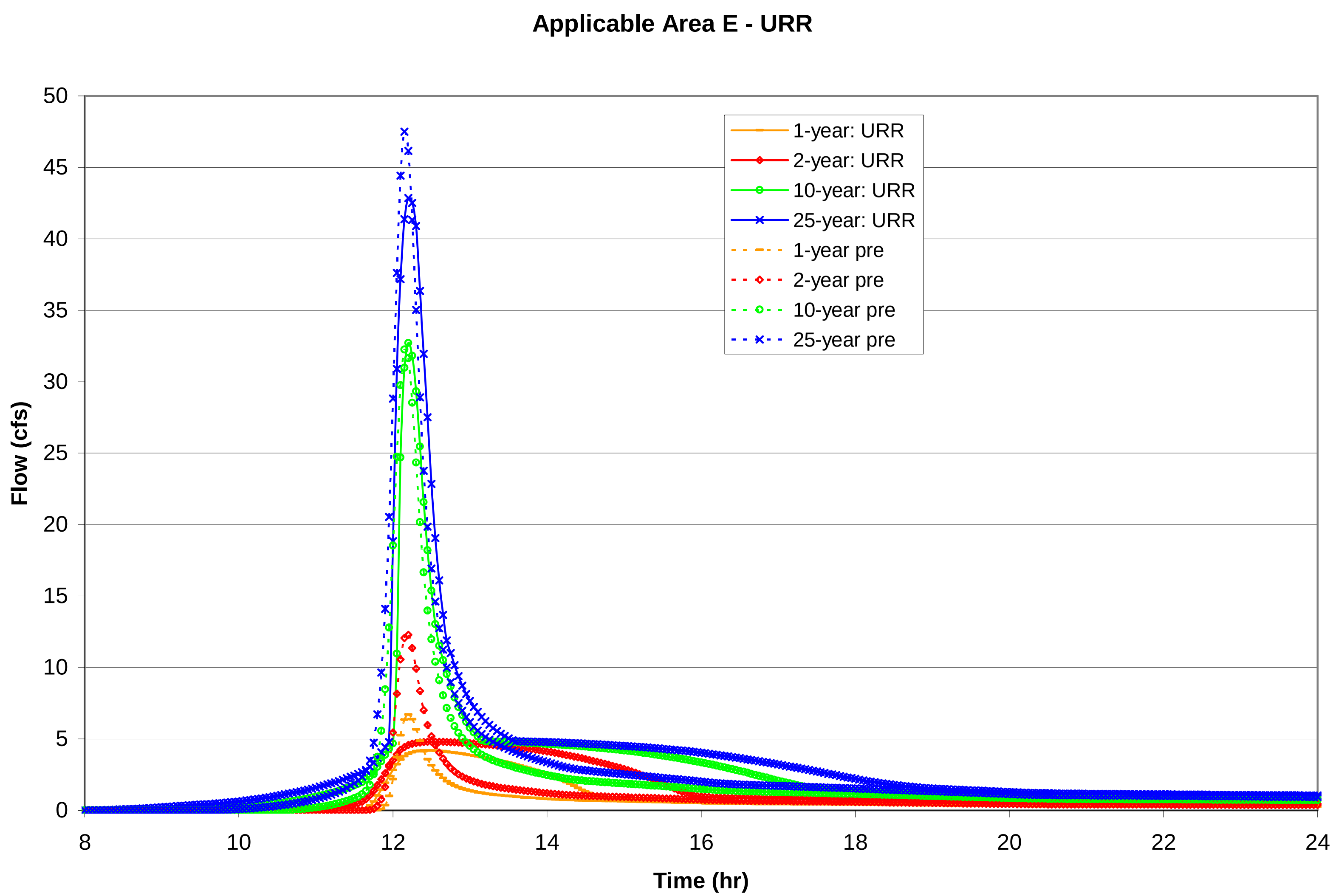
L-5. Post-development hydrographs for applicable area B with the Unit Release Rate design. Predevelopment hydrographs are provided for comparison.



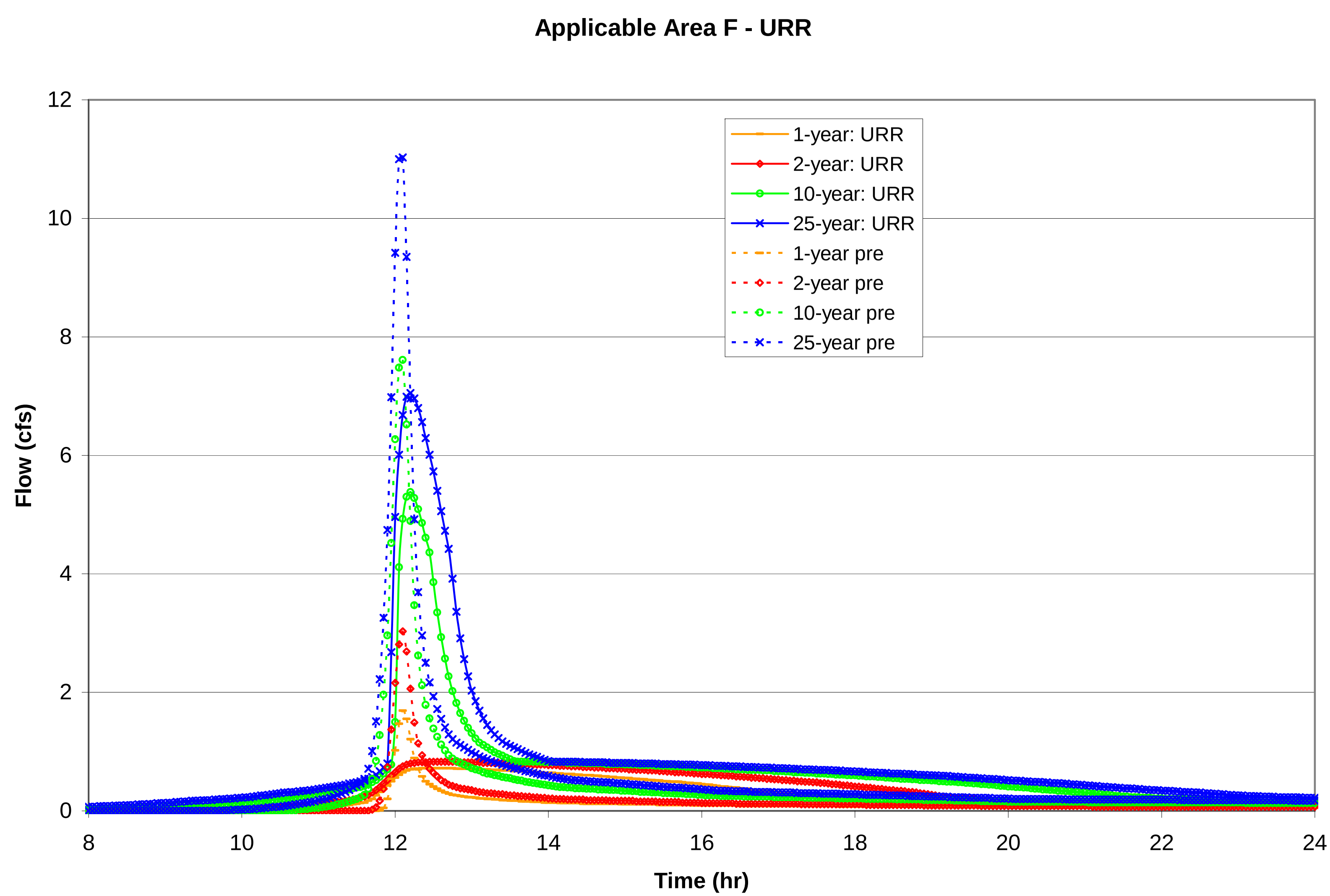
L-6. Post-development hydrographs for applicable area C with the Unit Release Rate design. Predevelopment hydrographs are provided for comparison.



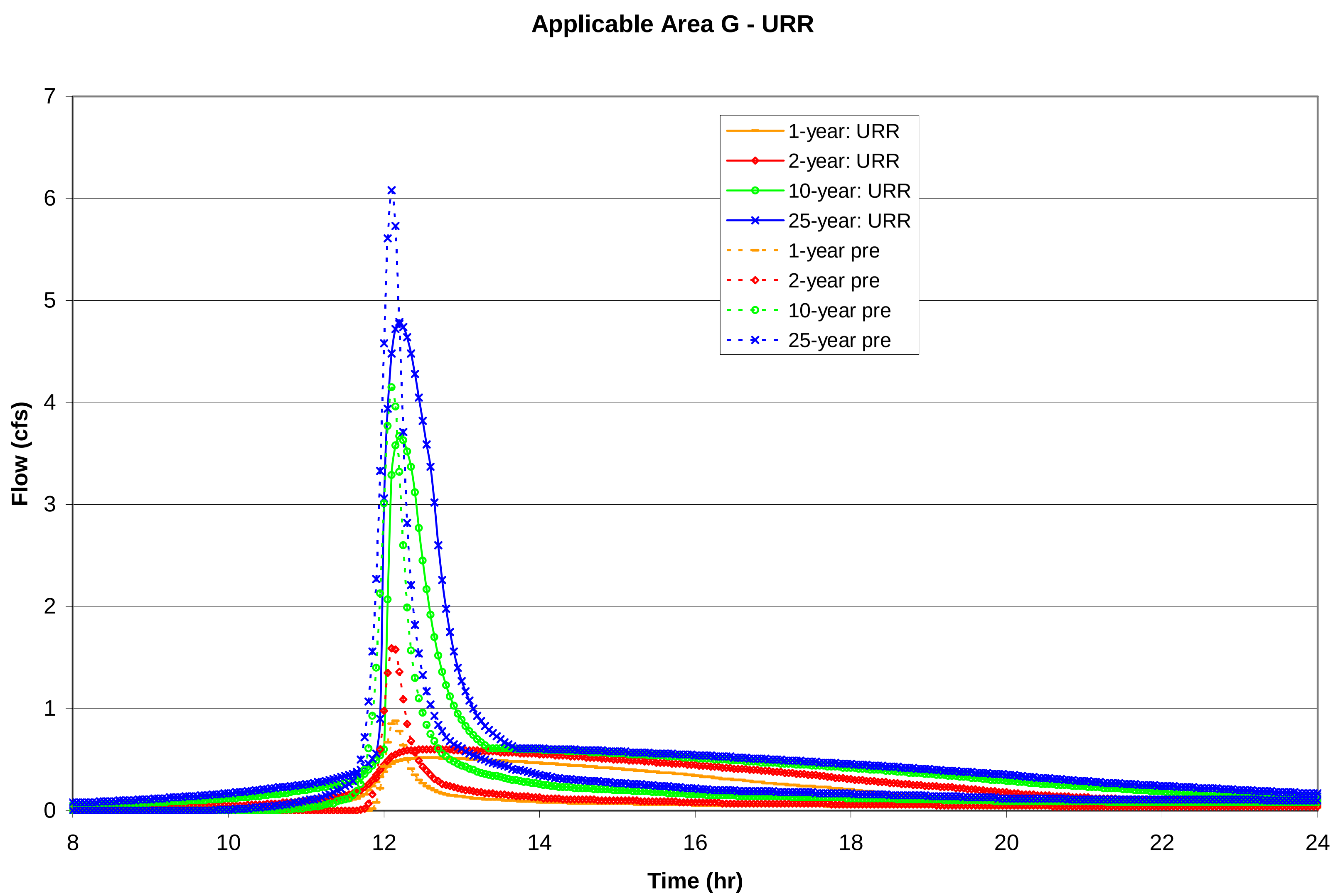
L-7. Post-development hydrographs for applicable area D with the Unit Release Rate design. Predevelopment hydrographs are provided for comparison.



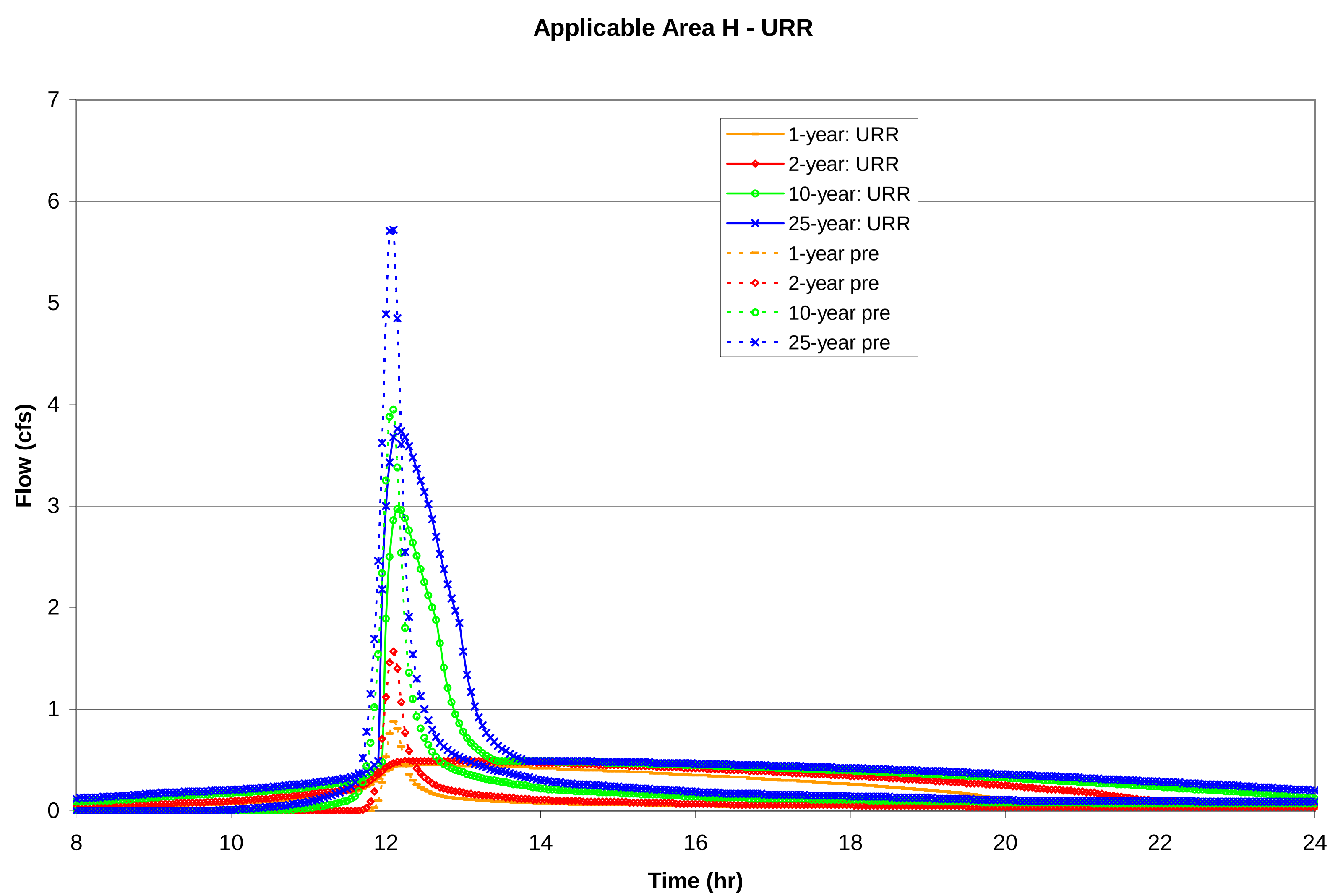
L-8. Post-development hydrographs for applicable area E with the Unit Release Rate design. Predevelopment hydrographs are provided for comparison.



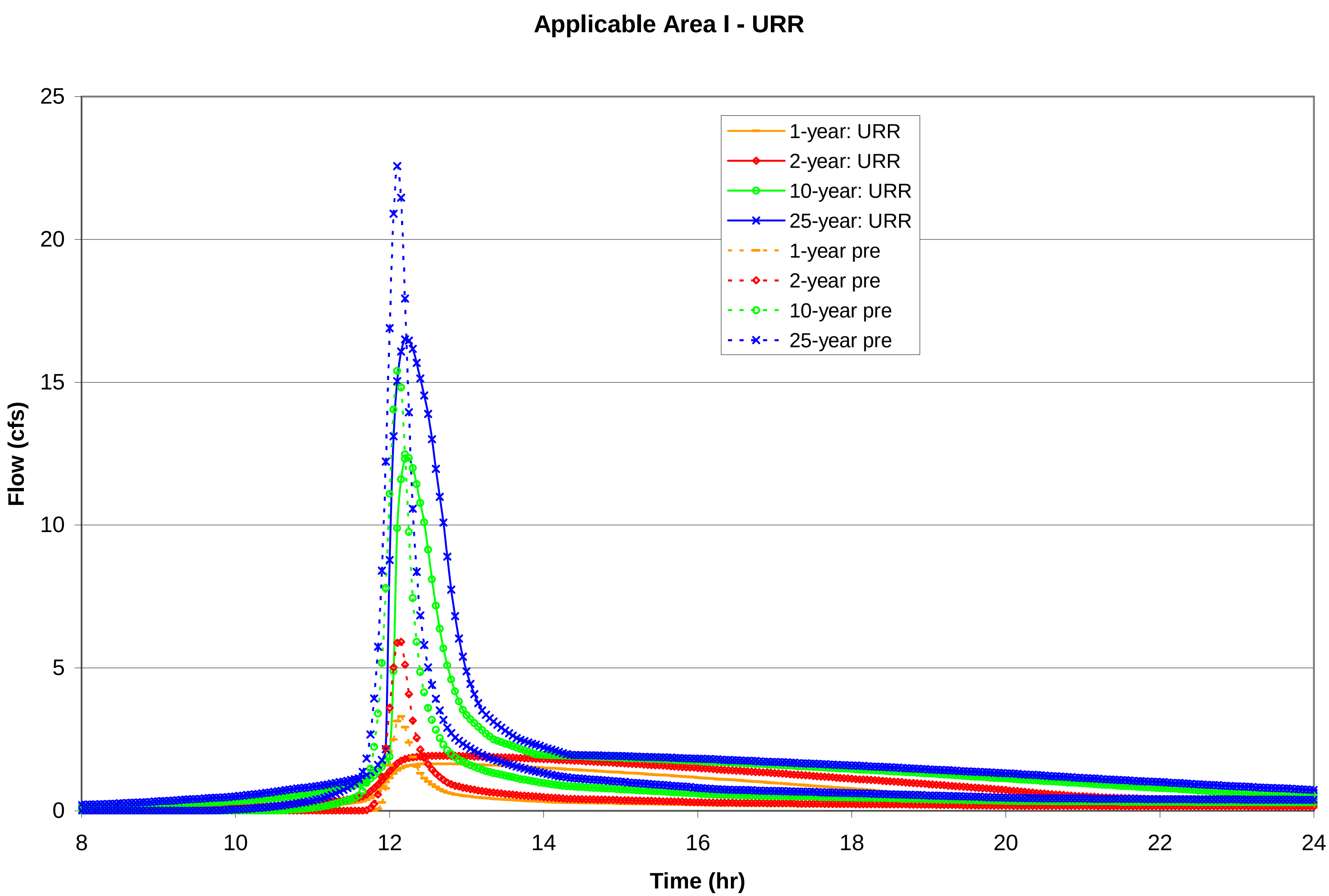
L-9. Post-development hydrographs for applicable area F with the Unit Release Rate design. Predevelopment hydrographs are provided for comparison.



L-10. Post-development hydrographs for applicable area G with the Unit Release Rate design. Predevelopment hydrographs are provided for comparison.



L-11. Post-development hydrographs for applicable area H with the Unit Release Rate design. Predevelopment hydrographs are provided for comparison.



L-12. Post-development hydrographs for applicable area I with the Unit Release Rate design. Predevelopment hydrographs are provided for comparison.

Appendix M: Information for the Stratified Release Rates strategy

- **M-1: Allowable release rates per applicable area for the 2-, 10- and 25-year storms.**
- **M-2: Control structure design information for achieving the Stratified Release Rate strategy allowable release rates.**
- **M-3: Post-development hydrographs for applicable area A with the Stratified Release Rate design. Predevelopment hydrographs are provided for comparison.**
- **M-4: Post-development hydrographs for applicable area A with the Stratified Release Rate design. Predevelopment hydrographs are provided for comparison.**
- **M-5: Post-development hydrographs for applicable area A with the Stratified Release Rate design. Predevelopment hydrographs are provided for comparison.**
- **M-6: Post-development hydrographs for applicable area A with the Stratified Release Rate design. Predevelopment hydrographs are provided for comparison.**
- **M-7: Post-development hydrographs for applicable area A with the Stratified Release Rate design. Predevelopment hydrographs are provided for comparison.**
- **M-8: Post-development hydrographs for applicable area A with the Stratified Release Rate design. Predevelopment hydrographs are provided for comparison.**
- **M-9: Post-development hydrographs for applicable area A with the Stratified Release Rate design. Predevelopment hydrographs are provided for comparison.**
- **M-10: Post-development hydrographs for applicable area A with the Stratified Release Rate design. Predevelopment hydrographs are provided for comparison.**

- **M-11: Post-development hydrographs for applicable area A with the Stratified Release Rate design. Predevelopment hydrographs are provided for comparison.**

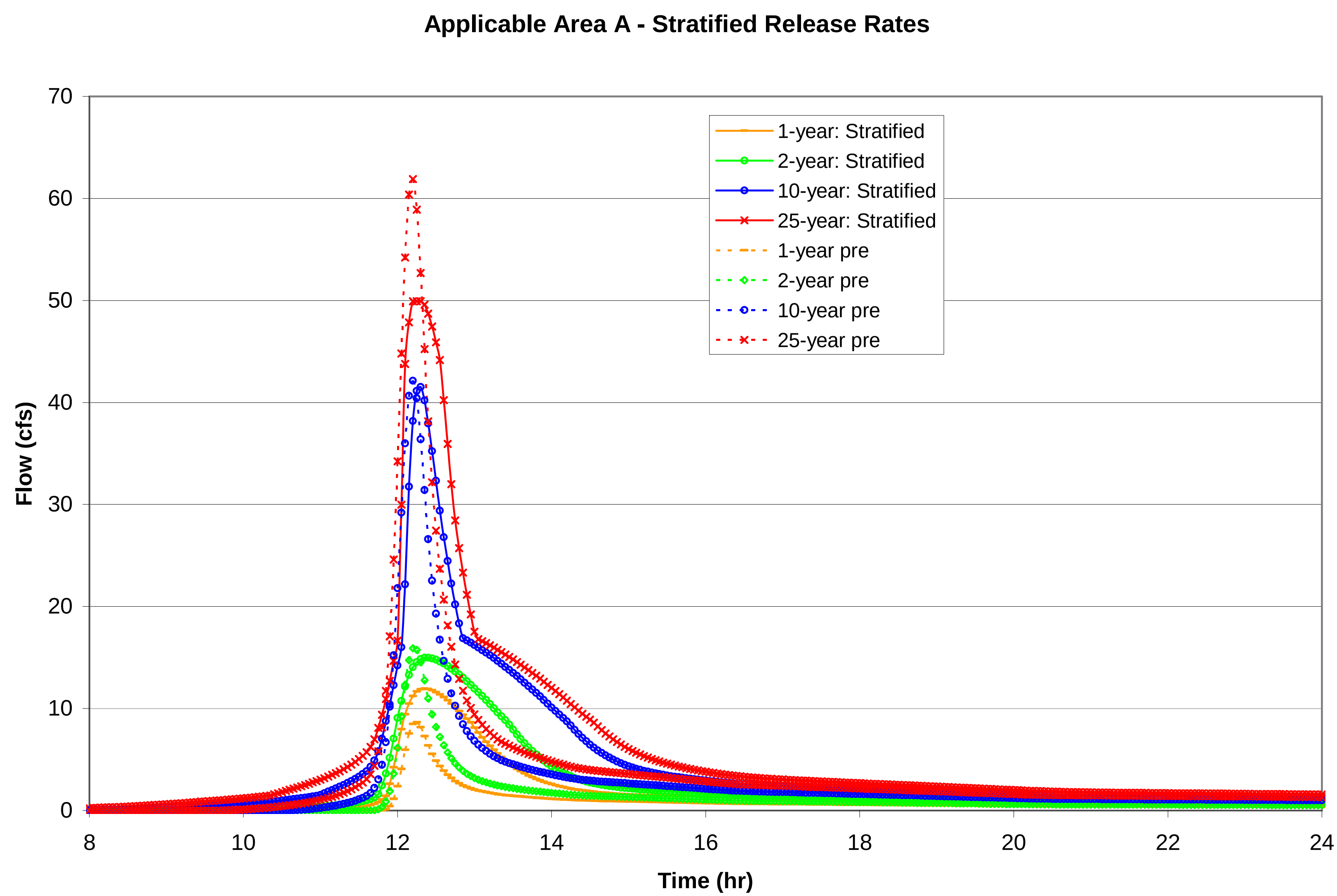
M-1. Allowable release rates per applicable area for the 2-, 10- and 25-year storms. Allowable release % and predevelopment runoff values determined as described in Section 3.2.3.3.

Applicable Area	Predevelopment Peak Runoff Rate (cfs)	Allowable release (% of predevelopment)	Allowable release rate (cfs)
2yr			
A	15.9	100	15.9
B	4.3	100	4.3
C	6.2	80	5.0
D	4.3	100	4.3
E	12.3	80	9.8
F	3.0	80	2.4
G	1.6	100	1.6
H	1.6	80	1.3
I	5.9	100	5.9
10yr			
A	42.1	100	42.1
B	11.0	100	11.0
C	15.7	80	12.6
D	11.1	100	11.1
E	32.3	100	32.3
F	7.6	80	6.1
G	4.2	80	3.4
H	4.0	60	2.4
I	15.4	100	15.4
25yr			
A	61.9	100	61.9
B	16.0	100	16.0
C	22.7	80	18.2
D	16.1	100	16.1
E	47.5	80	38.0
F	11.0	90	9.9
G	6.1	90	5.5
H	5.7	70	4.0
I	22.6	100	22.6

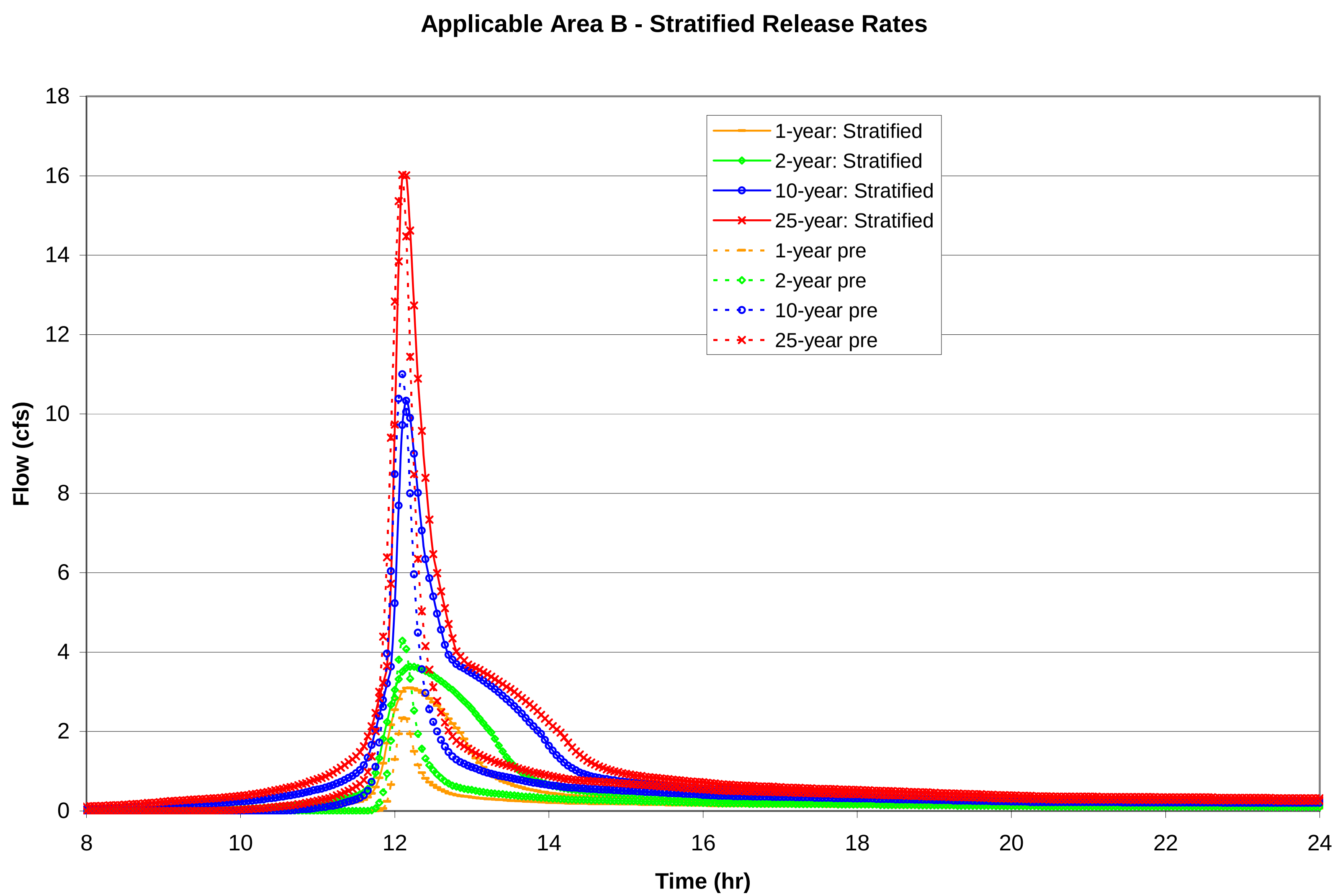
M-2. Control structure design information for achieving the Stratified Release Rate strategy allowable release rates. The primary orifice is generally for detaining the 2-year storm to the allowable release rate, the secondary orifice for the 10-year storm and the riser for the 25-year storm. Where a riser is not provided, the secondary orifice served the allowable release function for the 25-year storm in addition to the 10-year storm.

	Primary Orifice		Secondary Orifice		Riser	
Applicable Area Served	Dia. (in)	Invert (ft)	Dia. (in)	Invert (ft)	Dia. (in)	Invert (ft)
A	19	2,165.00	N/A		30	2,170.00
B	9	2,238.10	18	2,241.50	10	2,242.90
C	10	2,204.00	16	2,207.40	15	2,209.00
D	12	2,160.00	16	2,162.50	N/A	
E	12	2,137.26	28	2,143.10	N/A	
F	7	2,121.30	10	2,124.50	8	2,126.10
G	7	2,110.50	10	2,112.50	6	2,113.40
H	5	2,094.20	5	2,099.00	4.5	2,100.80
I	13	2,078.50	2'Wx 1'H*	2,080.80	21	2,081.80

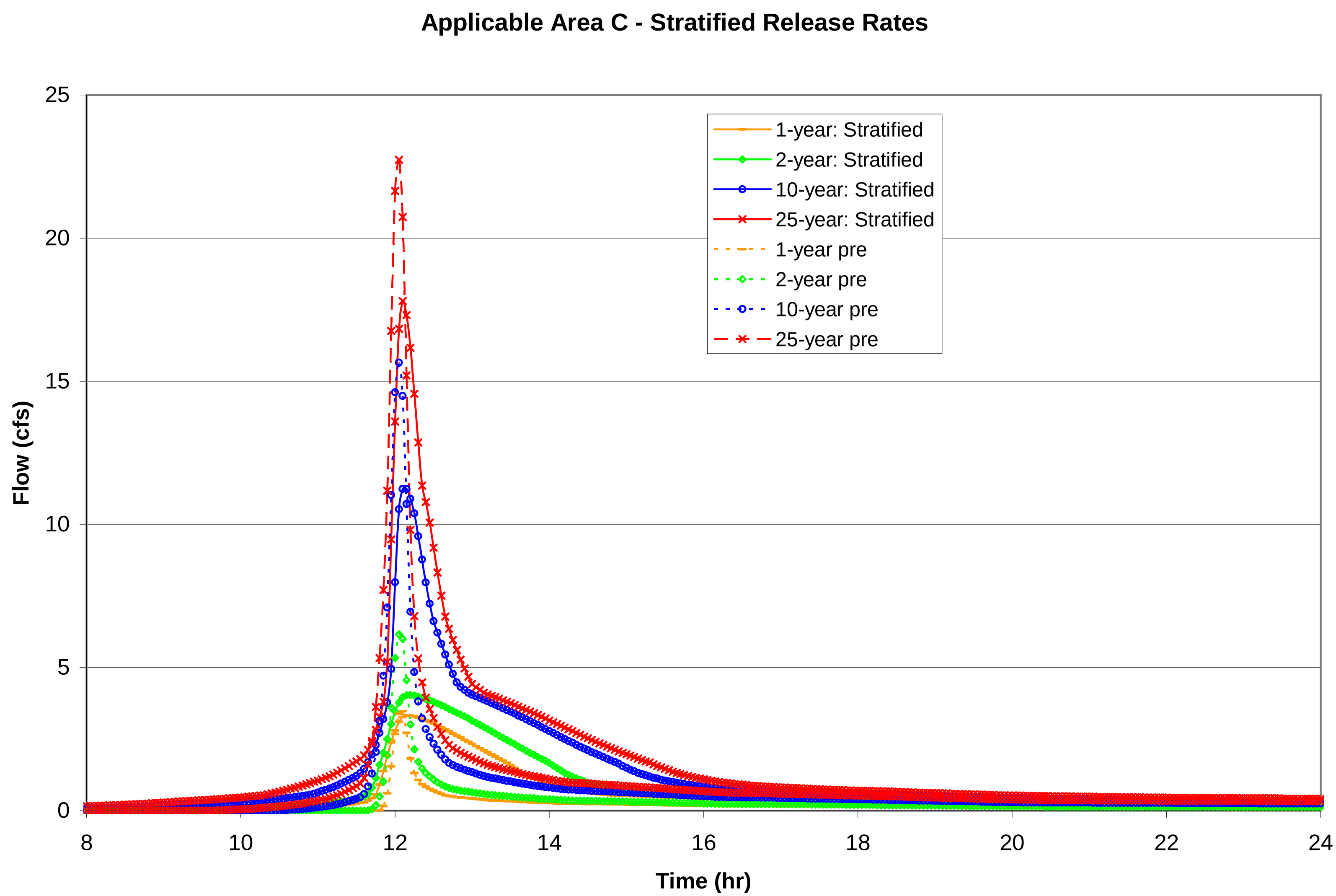
* A rectangular orifice was used instead of circular, dimensions are provided



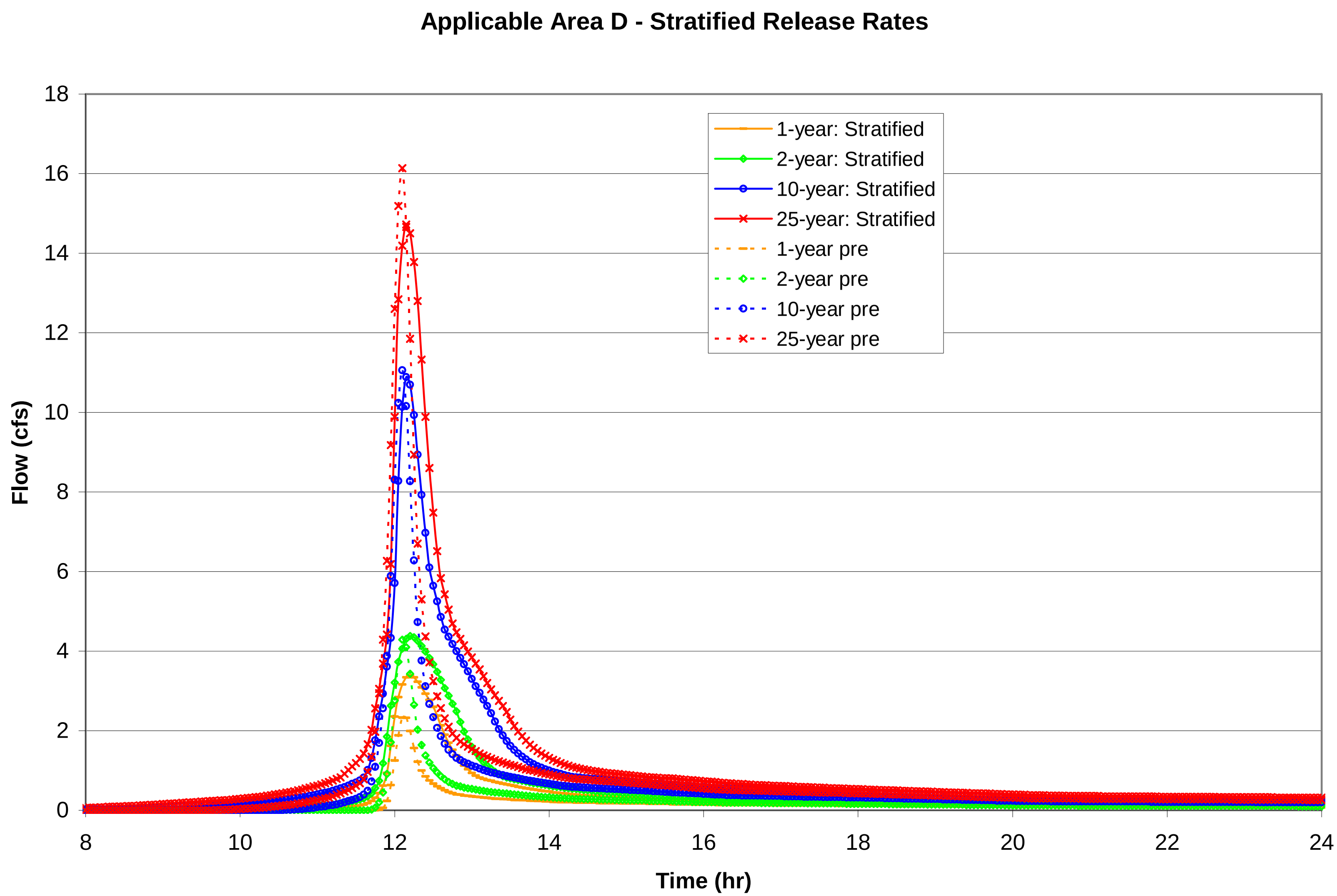
M-3. Post-development hydrographs for applicable area A with the Stratified Release Rate design. Predevelopment hydrographs are provided for comparison.



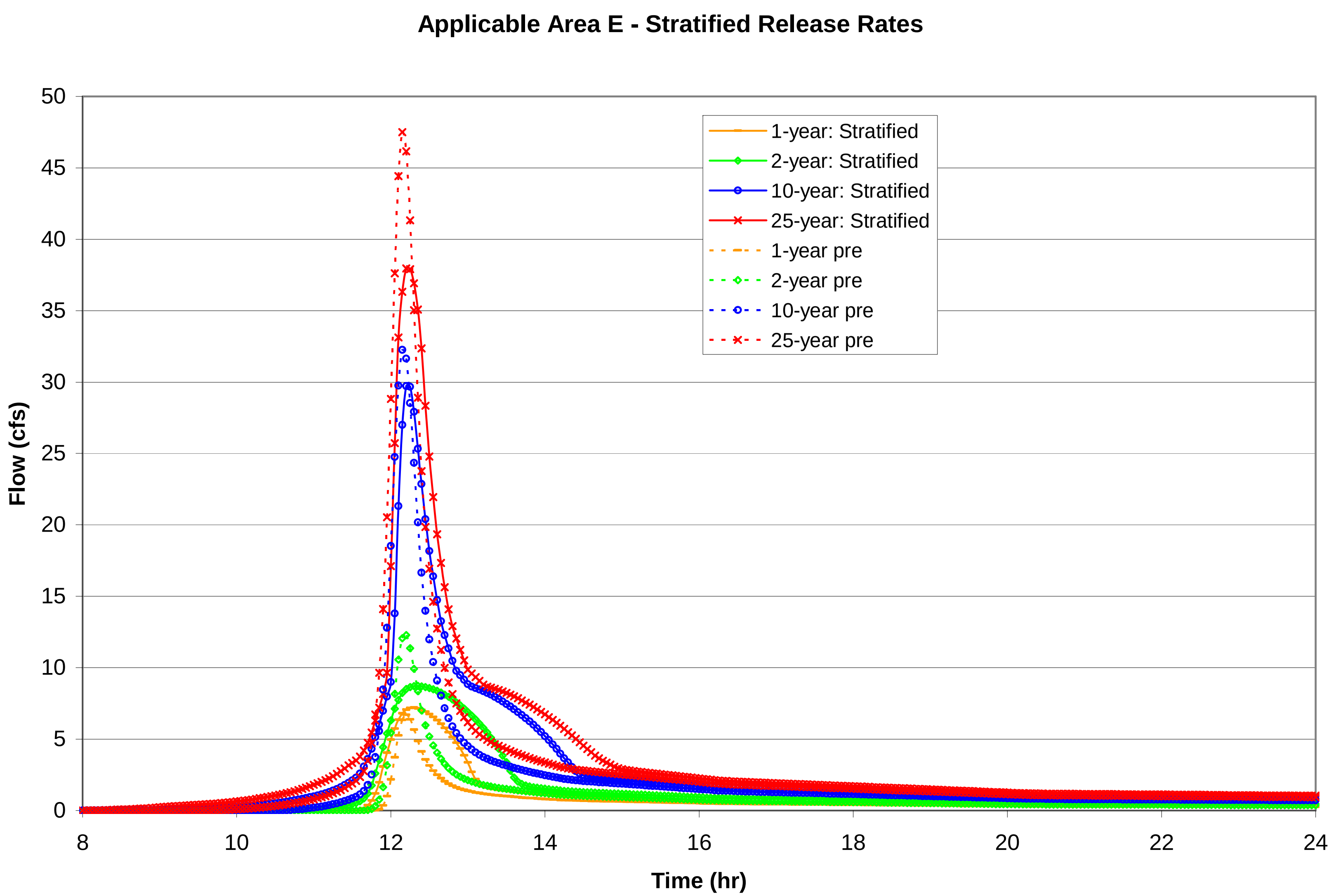
M-4. Post-development hydrographs for applicable area B with the Stratified Release Rate design. Predevelopment hydrographs are provided for comparison.



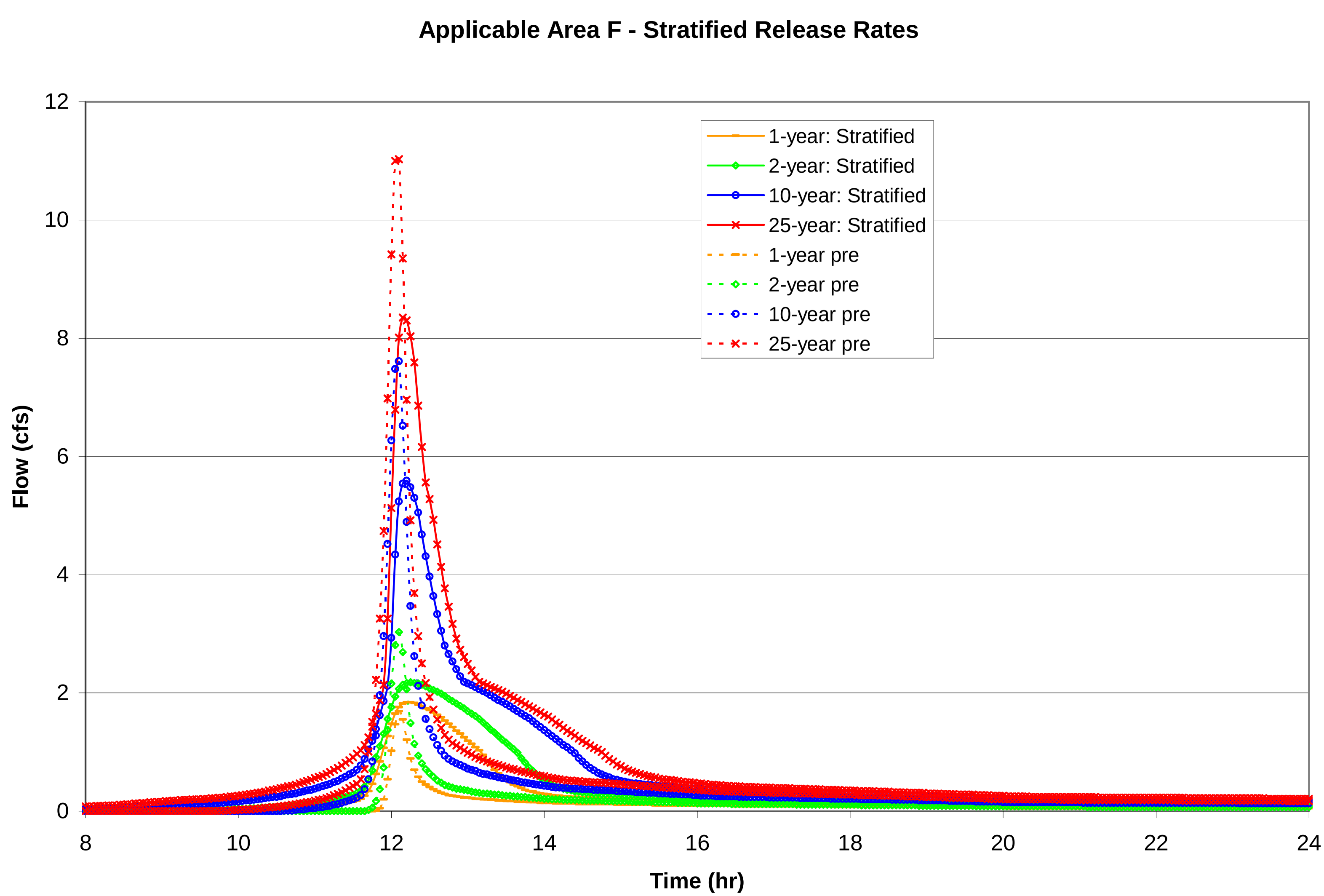
M-5. Post-development hydrographs for applicable area C with the Stratified Release Rate design. Predevelopment hydrographs are provided for comparison.



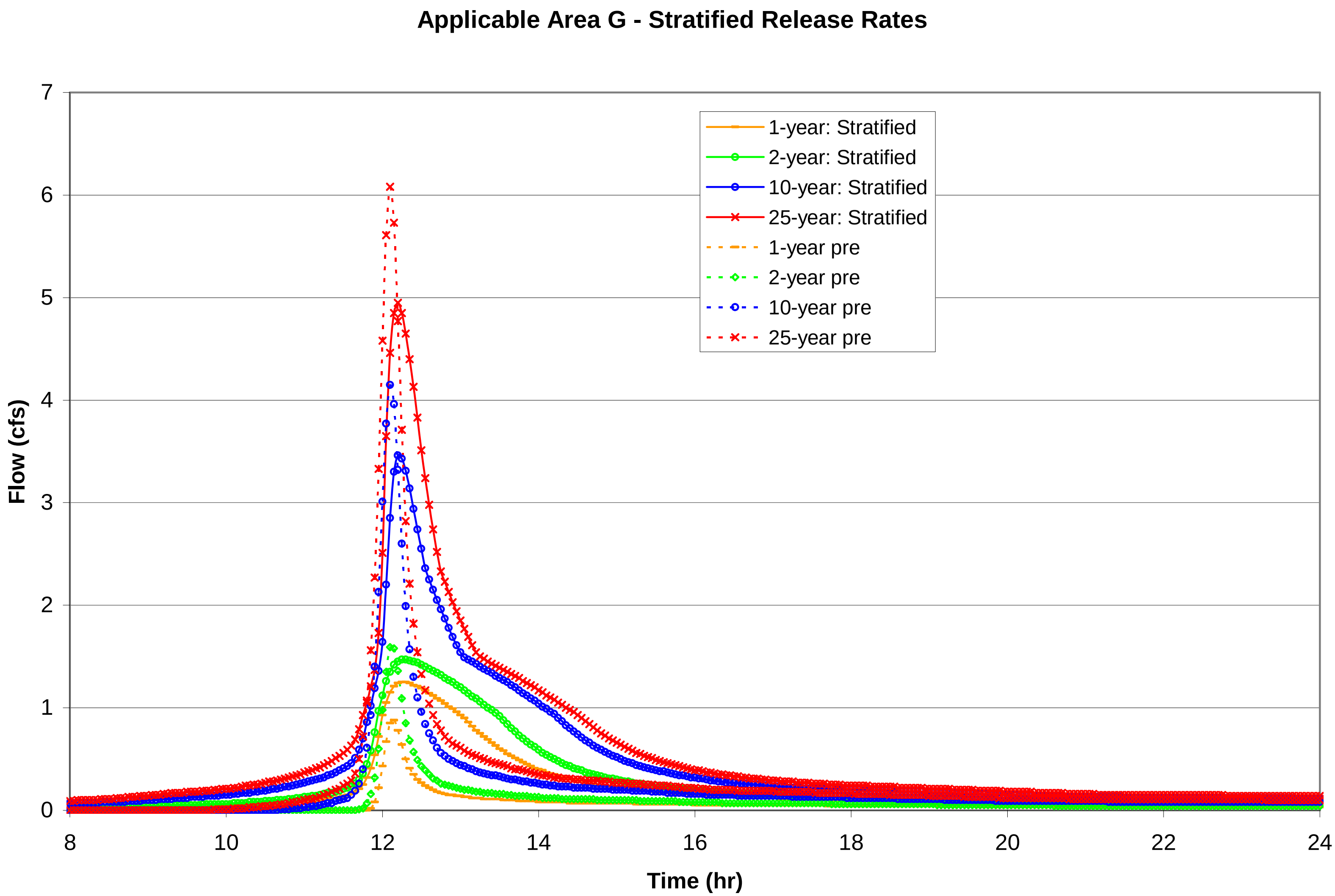
M-6. Post-development hydrographs for applicable area D with the Stratified Release Rate design. Predevelopment hydrographs are provided for comparison.



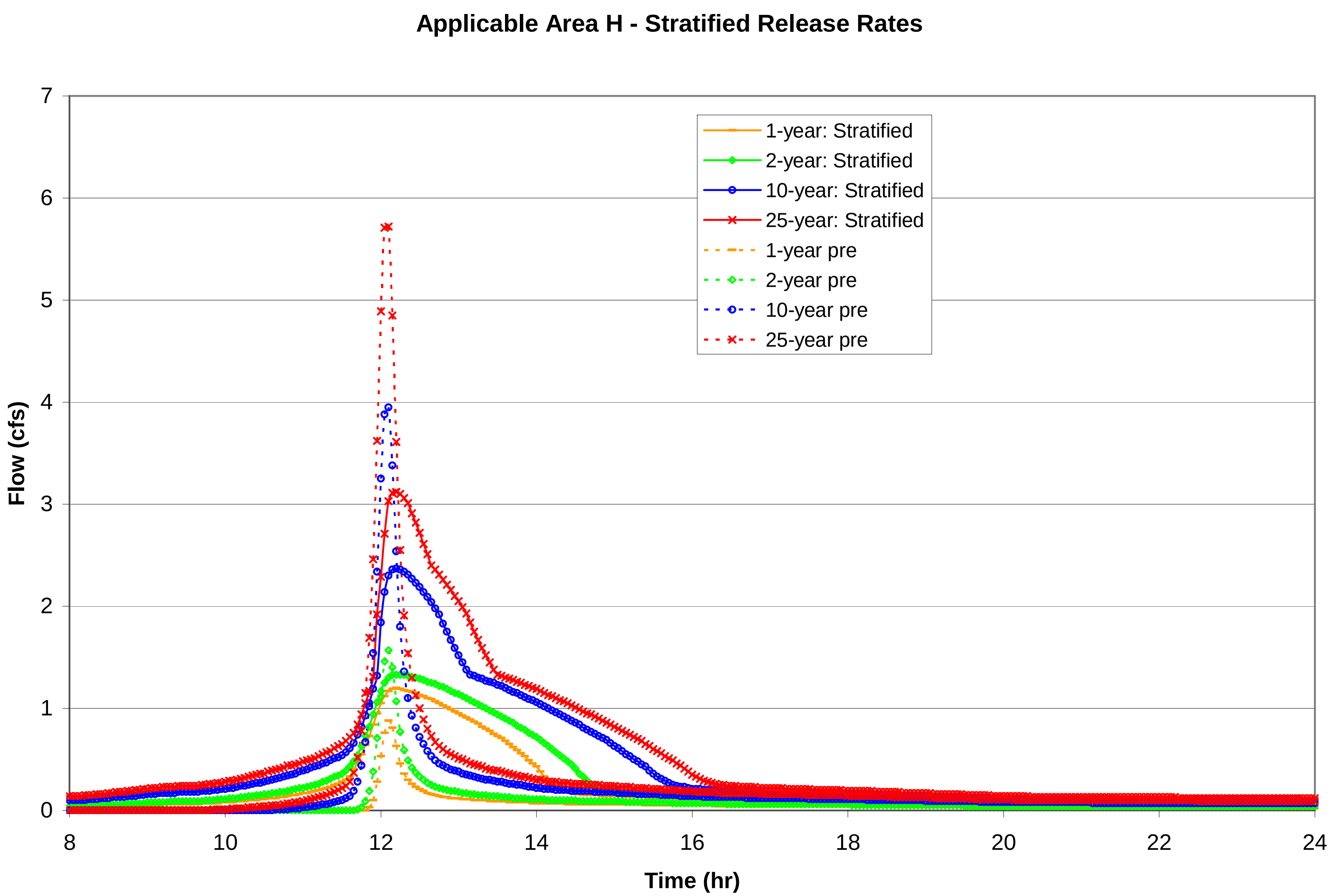
M-7. Post-development hydrographs for applicable area E with the Stratified Release Rate design. Predevelopment hydrographs are provided for comparison.



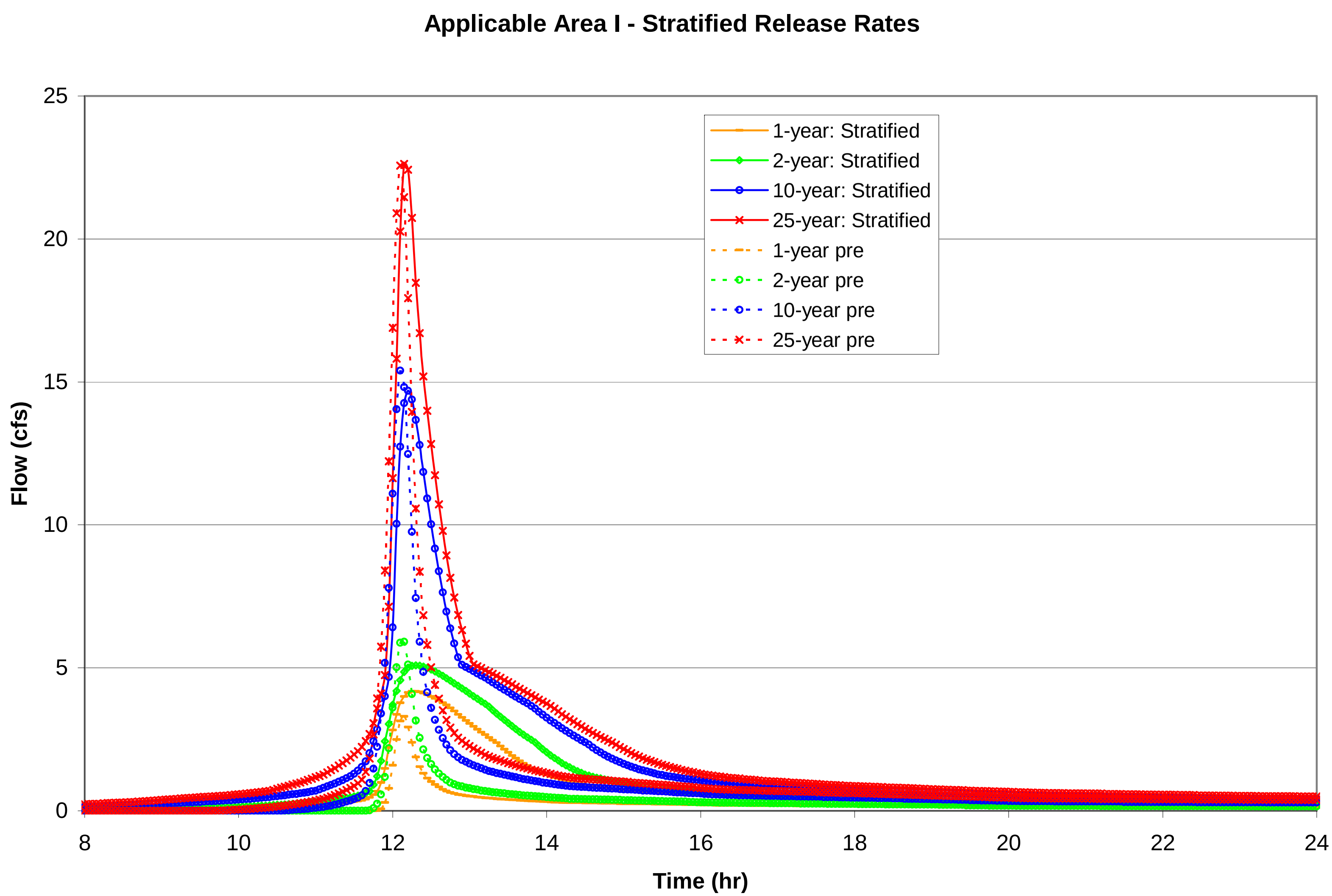
M-8. Post-development hydrographs for applicable area F with the Stratified Release Rate design. Predevelopment hydrographs are provided for comparison.



M-9. Post-development hydrographs for applicable area G with the Stratified Release Rate design. Predevelopment hydrographs are provided for comparison.



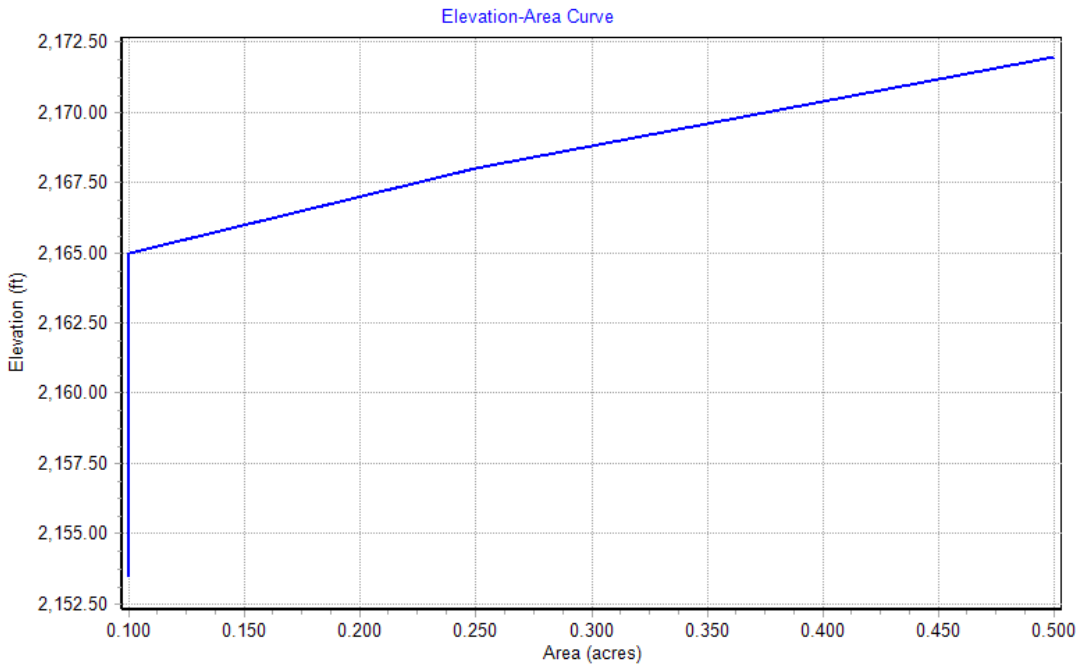
M-10. Post-development hydrographs for applicable area H with the Stratified Release Rate design. Predevelopment hydrographs are provided for comparison.



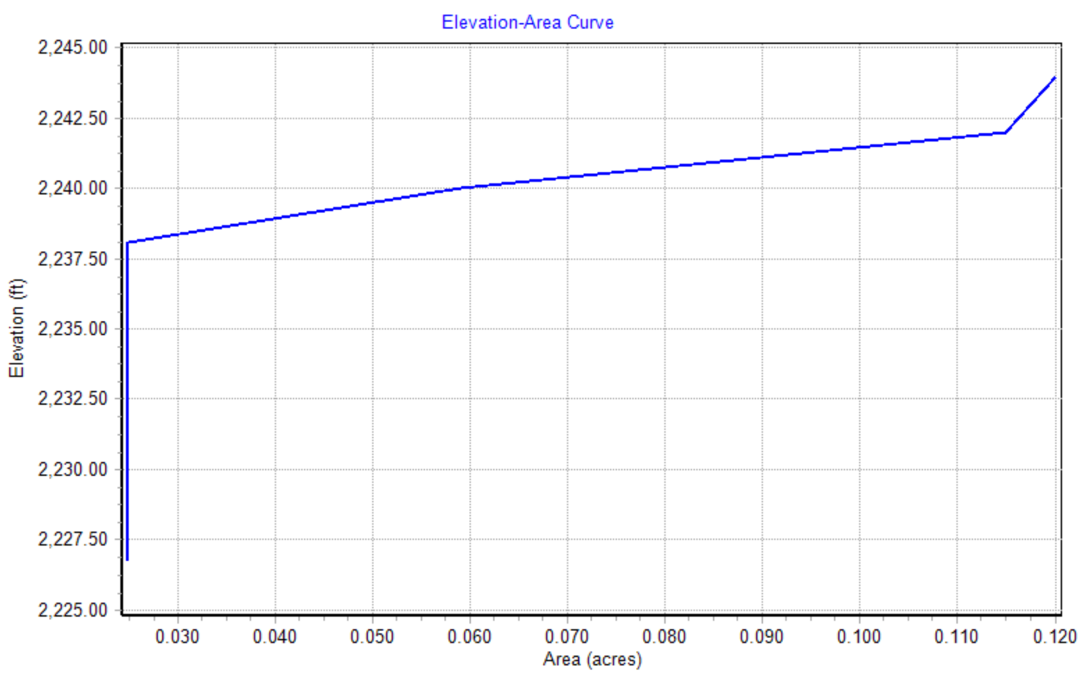
M-11. Post-development hydrographs for applicable area I with the Stratified Release Rate design. Predevelopment hydrographs are provided for comparison.

Appendix N: Information for the Low Impact Development strategy

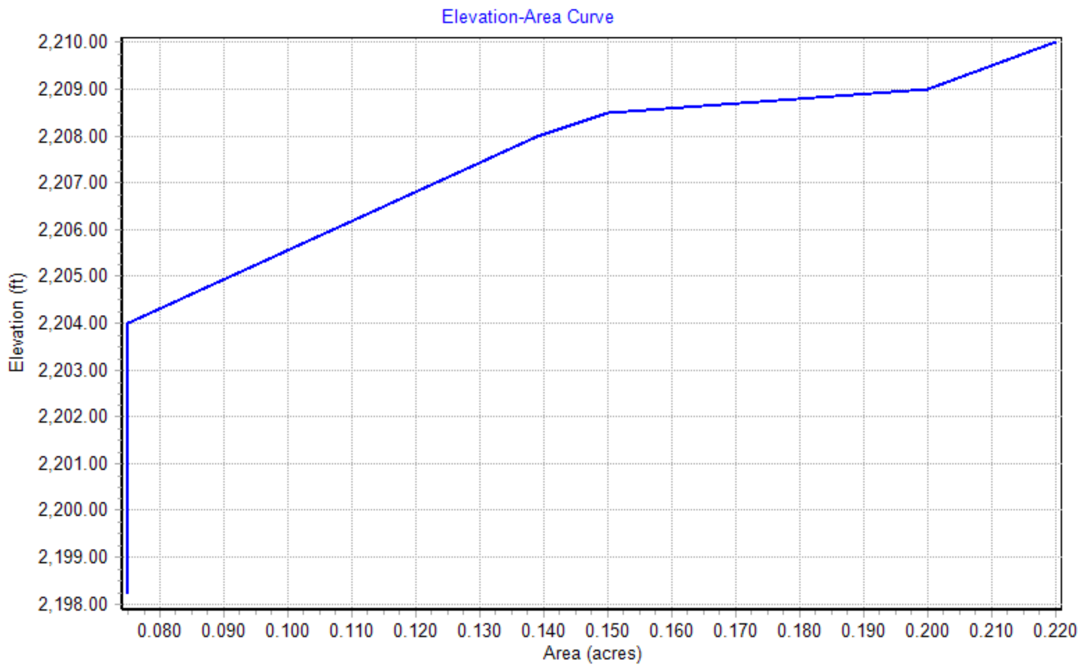
- **N-1: Elevation-area information for facility serving Applicable Area A.**
- **N-2: Elevation-area information for facility serving Applicable Area B.**
- **N-3: Elevation-area information for facility serving Applicable Area C.**
- **N-4: Elevation-area information for facility serving Applicable Area D.**
- **N-5: Elevation-area information for facility serving Applicable Area E.**
- **N-6: Elevation-area information for facility serving Applicable Area F.**
- **N-7: Elevation-area information for facility serving Applicable Area G.**
- **N-8: Elevation-area information for facility serving Applicable Area H.**
- **N-9: Elevation-area information for facility serving Applicable Area I.**
- **N-10: Control structure design information for achieving the Low Impact Development strategy objectives.**
- **N-11: Pre- and post-development hydrographs for applicable area A with the traditional with water quality design.**
- **N-11: Pre- and post-development hydrographs for applicable area B with the traditional with water quality design.**
- **N-11: Pre- and post-development hydrographs for applicable area C with the traditional with water quality design.**
- **N-11: Pre- and post-development hydrographs for applicable area D with the traditional with water quality design.**
- **N-11: Pre- and post-development hydrographs for applicable area E with the traditional with water quality design.**
- **N-11: Pre- and post-development hydrographs for applicable area F with the traditional with water quality design.**
- **N-11: Pre- and post-development hydrographs for applicable area G with the traditional with water quality design.**
- **N-11: Pre- and post-development hydrographs for applicable area H with the traditional with water quality design.**
- **N-11: Pre- and post-development hydrographs for applicable area I with the traditional with water quality design.**



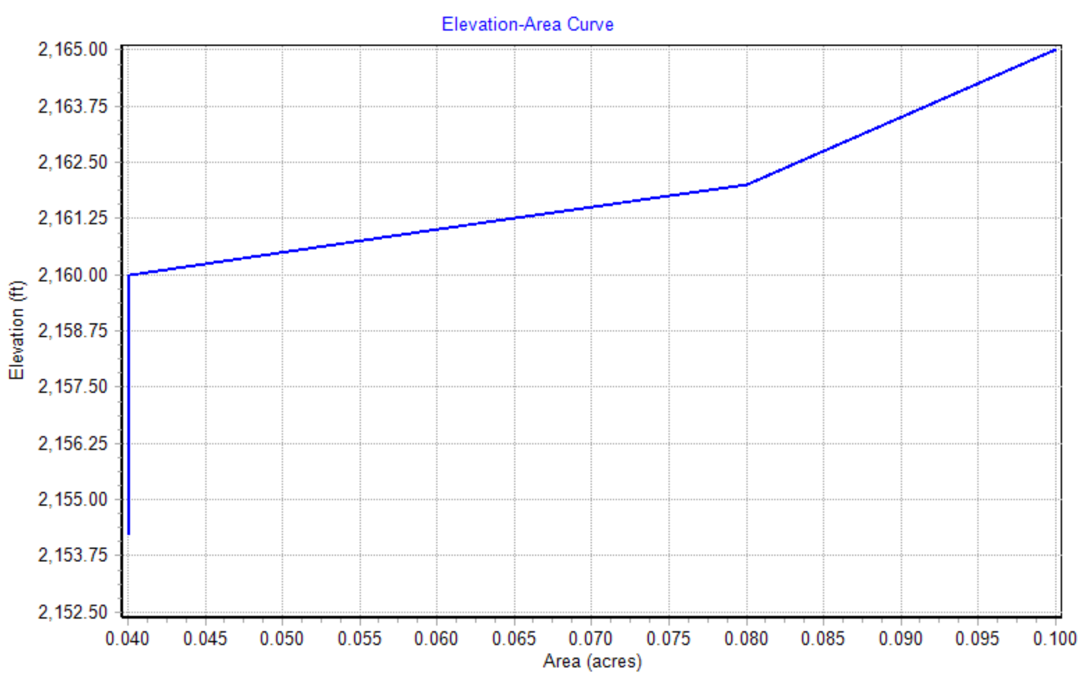
N-1. Elevation-area information for facility serving Applicable Area A. Storage is provided beneath the primary orifice to provide storage that simulates the reduction in volume specified with the LID strategy.



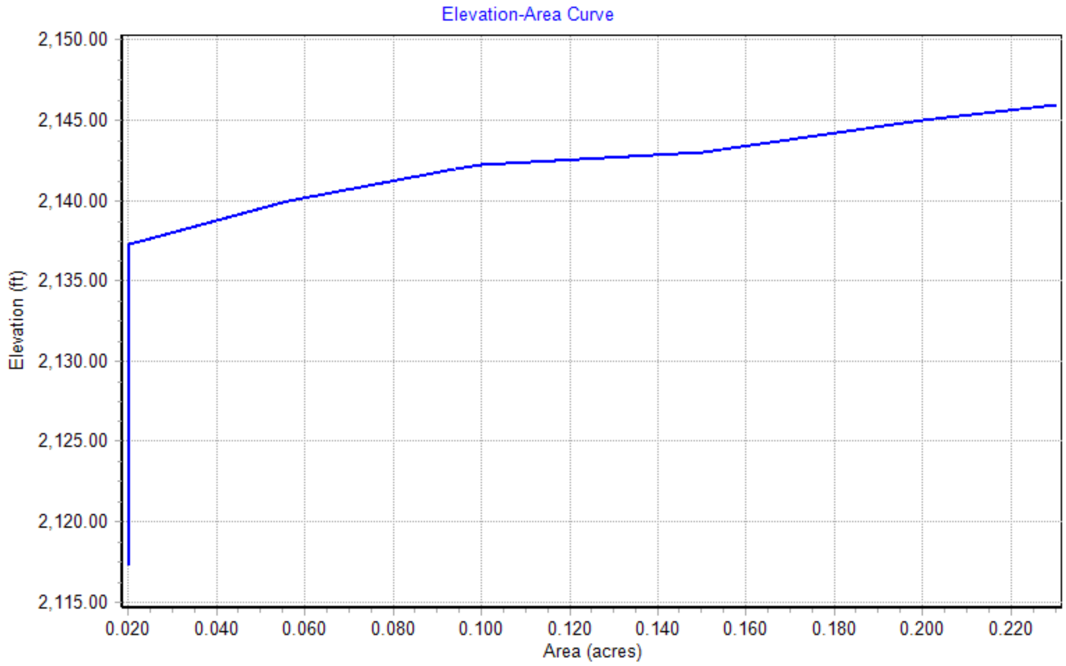
N-2. Elevation-area information for facility serving Applicable Area B. Storage is provided beneath the primary orifice to provide storage that simulates the reduction in volume specified with the LID strategy.



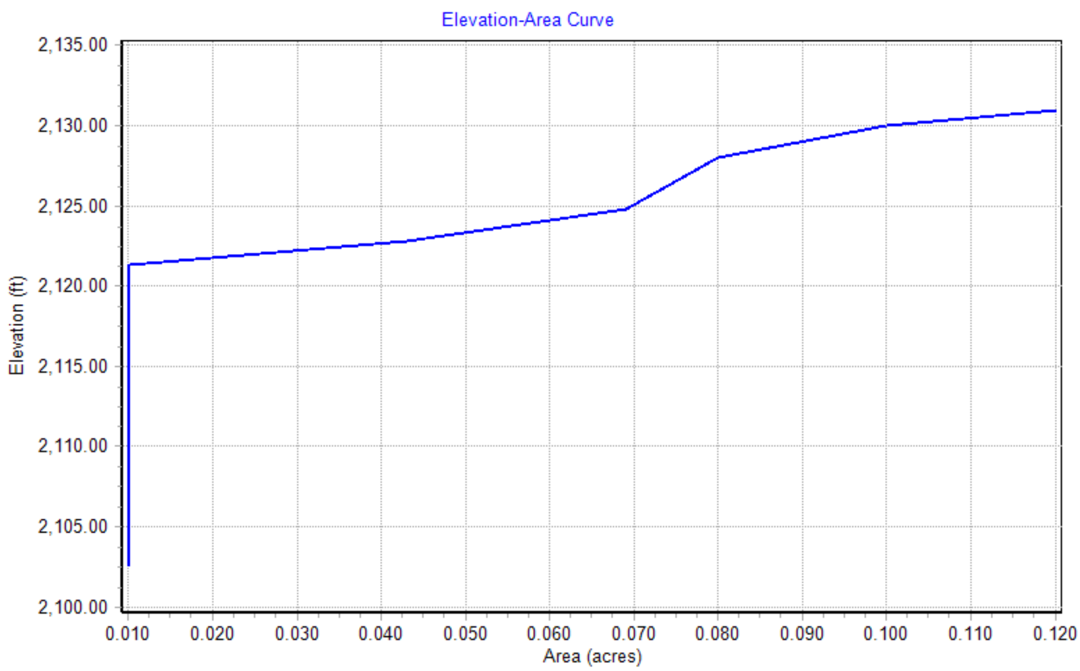
N-3. Elevation-area information for facility serving Applicable Area C. Storage is provided beneath the primary orifice to provide storage that simulates the reduction in volume specified with the LID strategy.



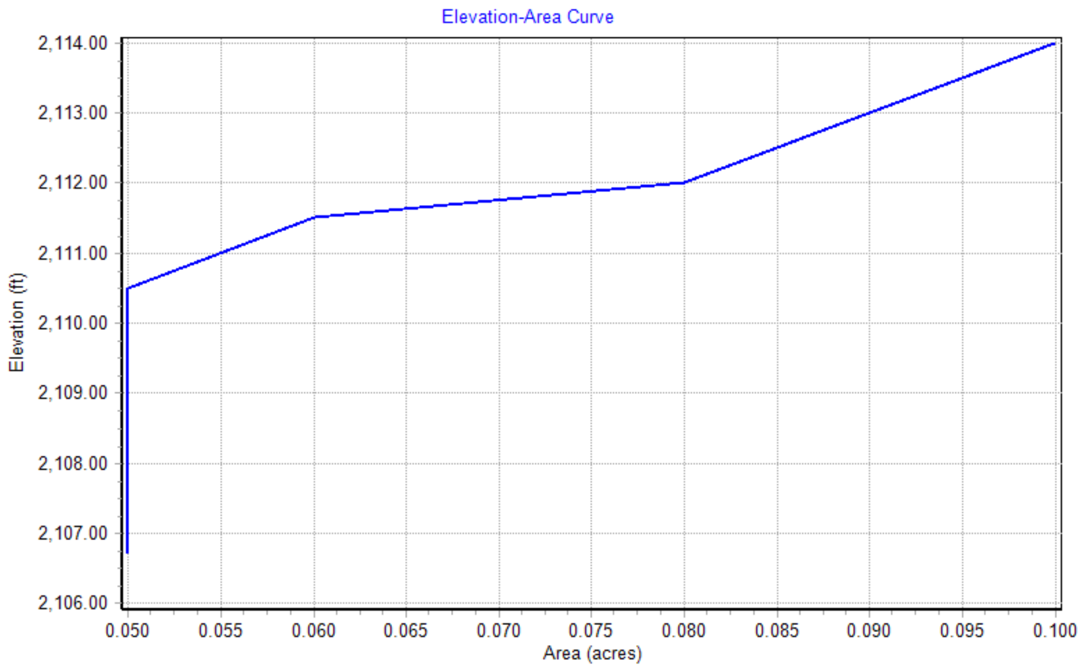
N-4. Elevation-area information for facility serving Applicable Area D. Storage is provided beneath the primary orifice to provide storage that simulates the reduction in volume specified with the LID strategy.



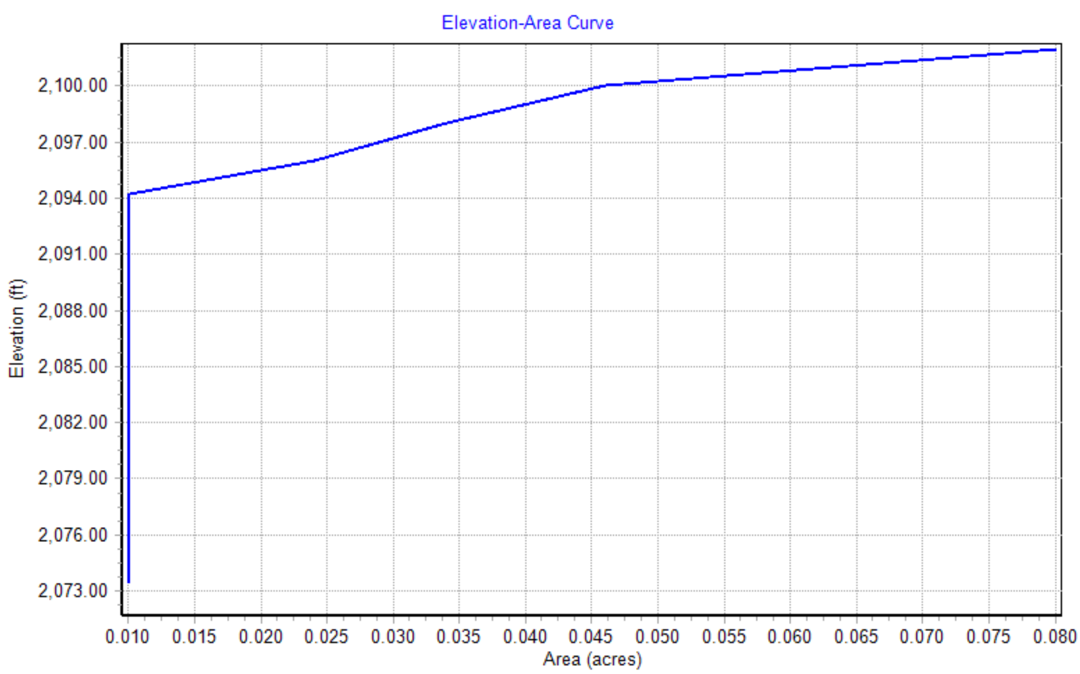
N-5. Elevation-area information for facility serving Applicable Area E. Storage is provided beneath the primary orifice to provide storage that simulates the reduction in volume specified with the LID strategy.



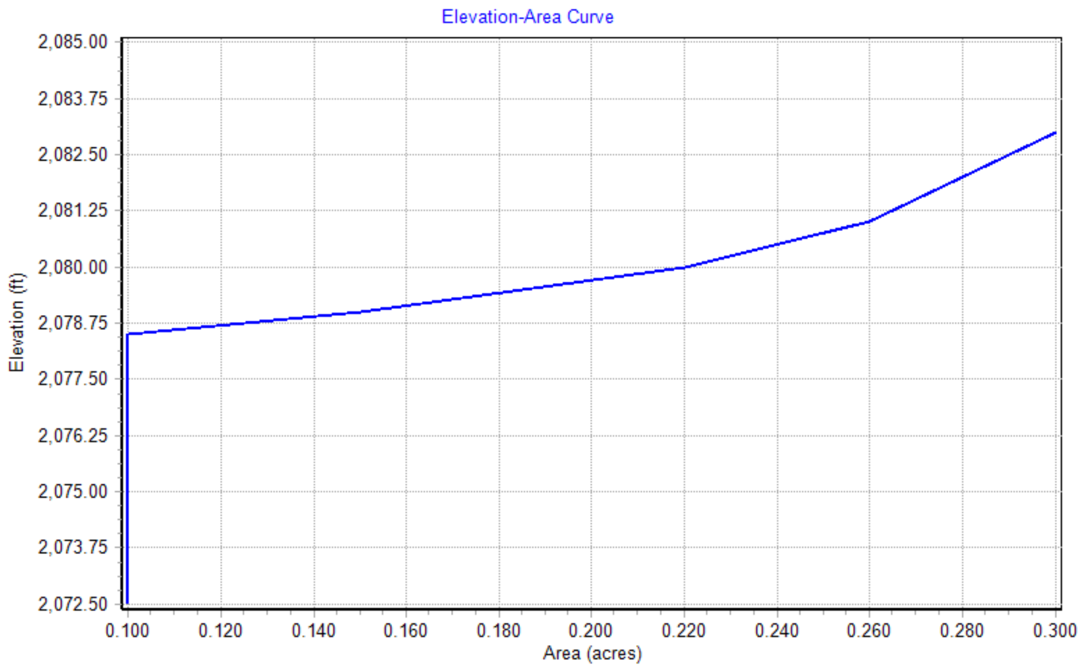
N-6. Elevation-area information for facility serving Applicable Area F. Storage is provided beneath the primary orifice to provide storage that simulates the reduction in volume specified with the LID strategy.



N-7. Elevation-area information for facility serving Applicable Area G. Storage is provided beneath the primary orifice to provide storage that simulates the reduction in volume specified with the LID strategy.



N-8. Elevation-area information for facility serving Applicable Area H. Storage is provided beneath the primary orifice to provide storage that simulates the reduction in volume specified with the LID strategy.



N-9. Elevation-area information for facility serving Applicable Area I. Storage is provided beneath the primary orifice to provide storage that simulates the reduction in volume specified with the LID strategy.

N-10. Control structure design information for achieving the Low Impact Development strategy objectives.

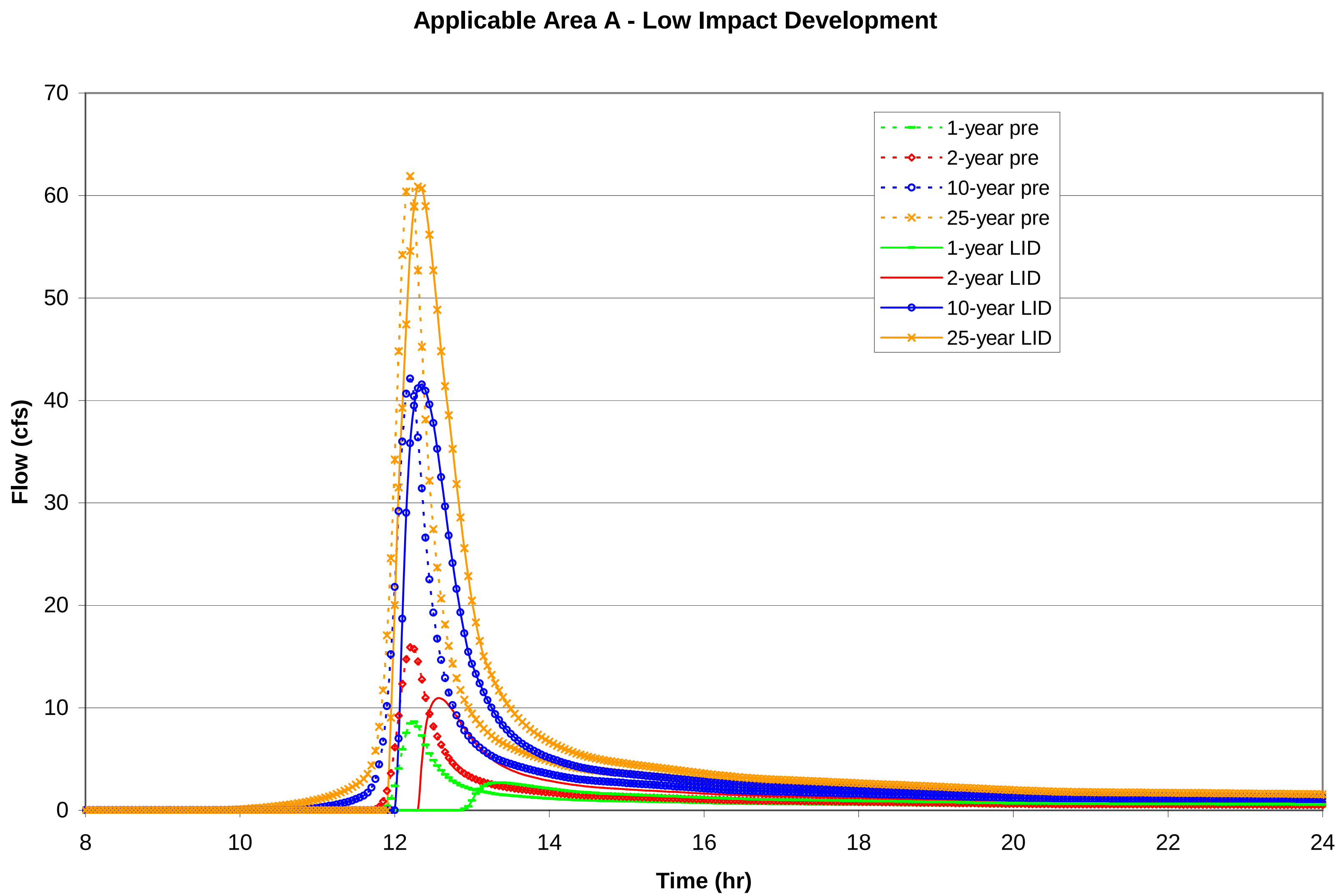
Facility Serving Applicable Area	Orifice		Secondary Orifice		Riser		Weir	
	Invert Elevation (ft)	Type/ Size* (in)	Crest Elevation (ft)	Type/ Size* (in)	Crest Elevation (ft)	Dia (ft)	Crest Elevation (ft)	Length (ft)
A	2,165.00	28 (2)	N/A		2,169.40	48	N/A	
B	2,238.80	12 (2)	N/A		2,241.50	48	N/A	
C	2,204.00	24	N/A		2,207.00	48	N/A	
D	2,160.00	1'H x 4'W (R)	N/A		N/A		2,163.00	15
E	2,137.26	18	2,140.10	17	2,143.50	48	N/A	
F	2,121.30	9 (2)	N/A		N/A		2,124.40	30
G	2,110.50	10 (2)	N/A		2,112.10	48	N/A	
H	2,094.20	7 (2)	N/A		2,097.90	36	N/A	
I	2,078.50	24 (2)	N/A		2,080.50	48	N/A	

* All orifices are circular unless an (R) is indicated for rectangular orifice, area of rectangular orifice is provided

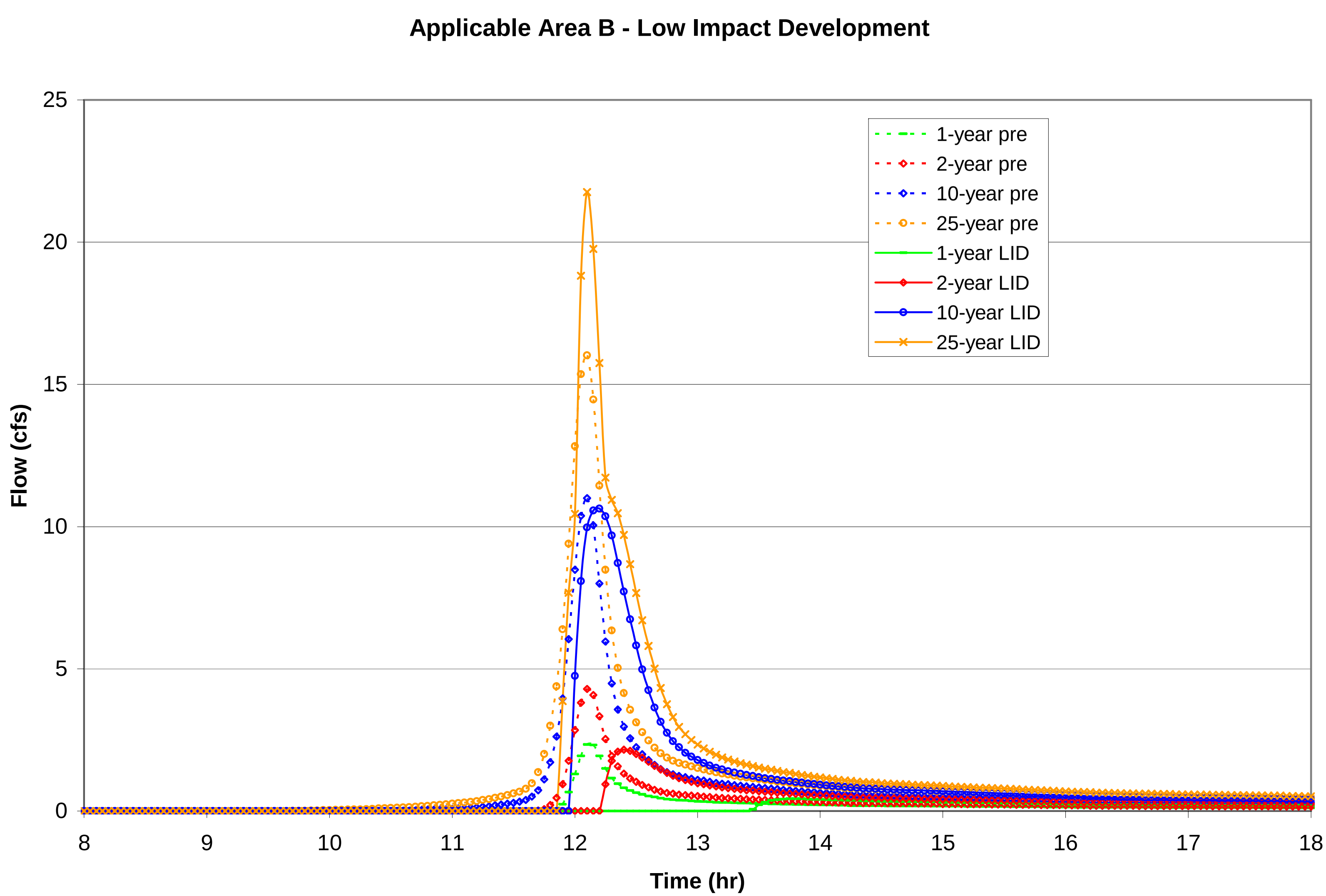
** Orifice coefficient = 0.6 in all cases

*** Weir coefficient = 3.1 in all cases

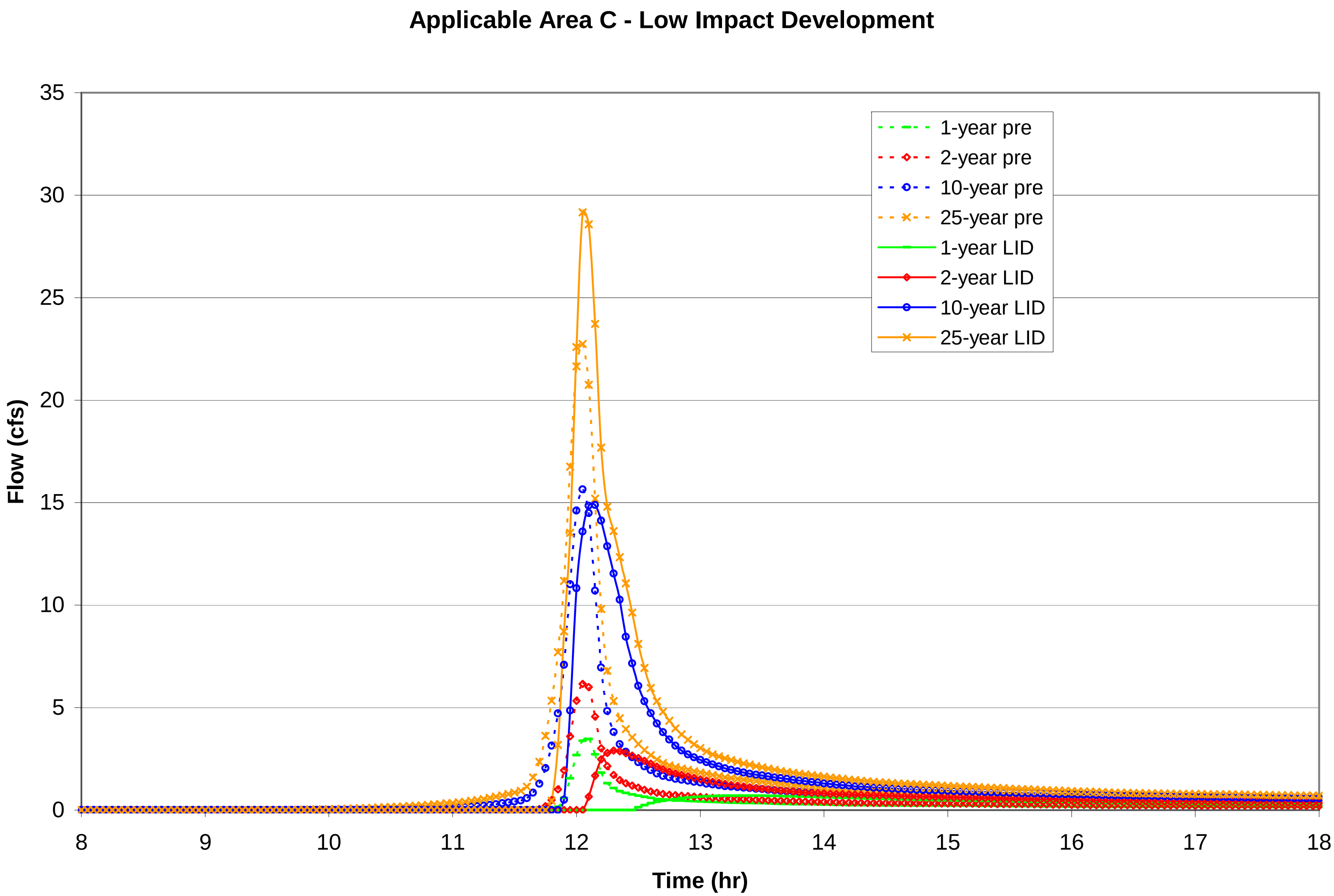
**** Weir is suppressed type in all cases



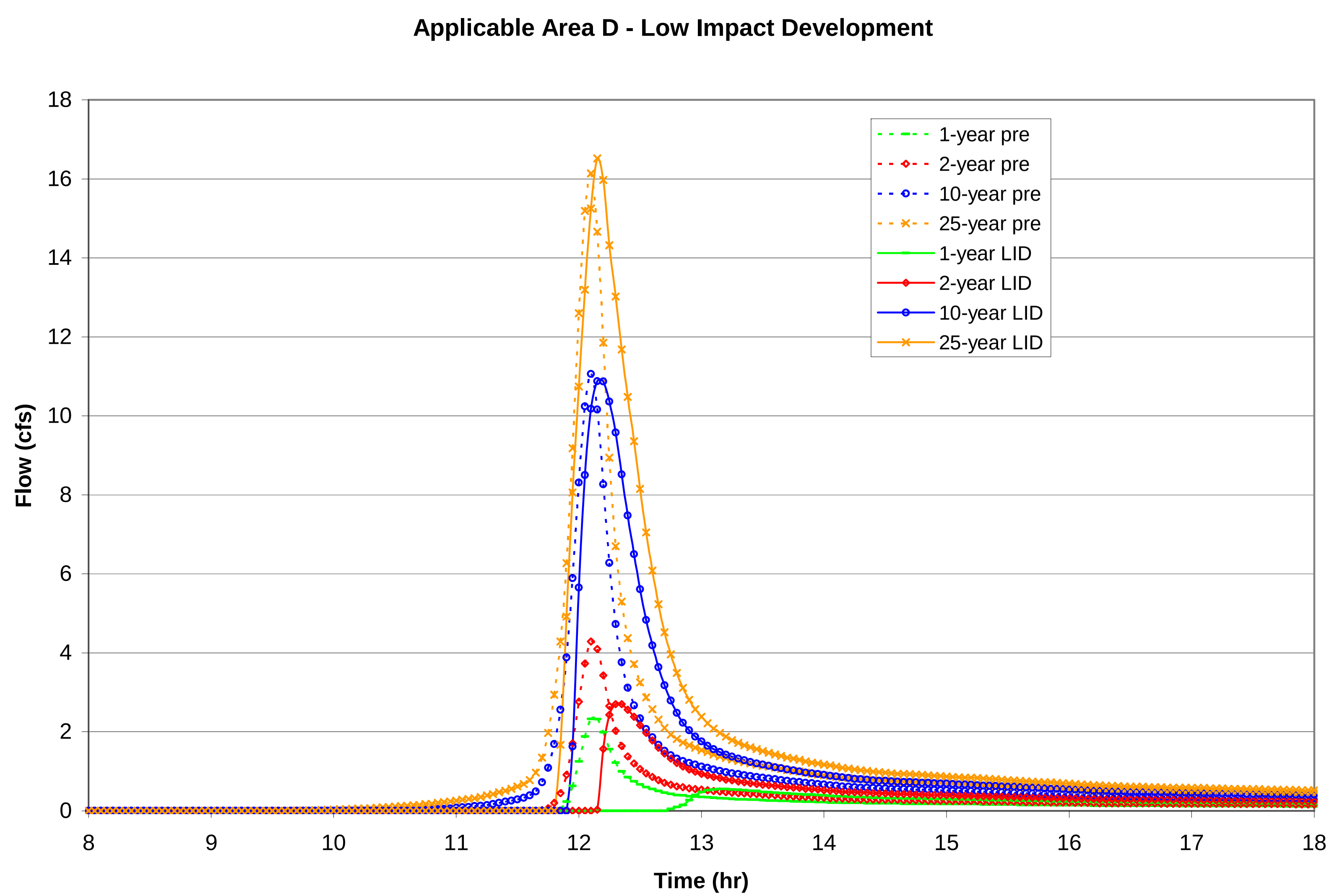
N-11. Pre- and post-development hydrographs for applicable area A with the traditional with water quality design.



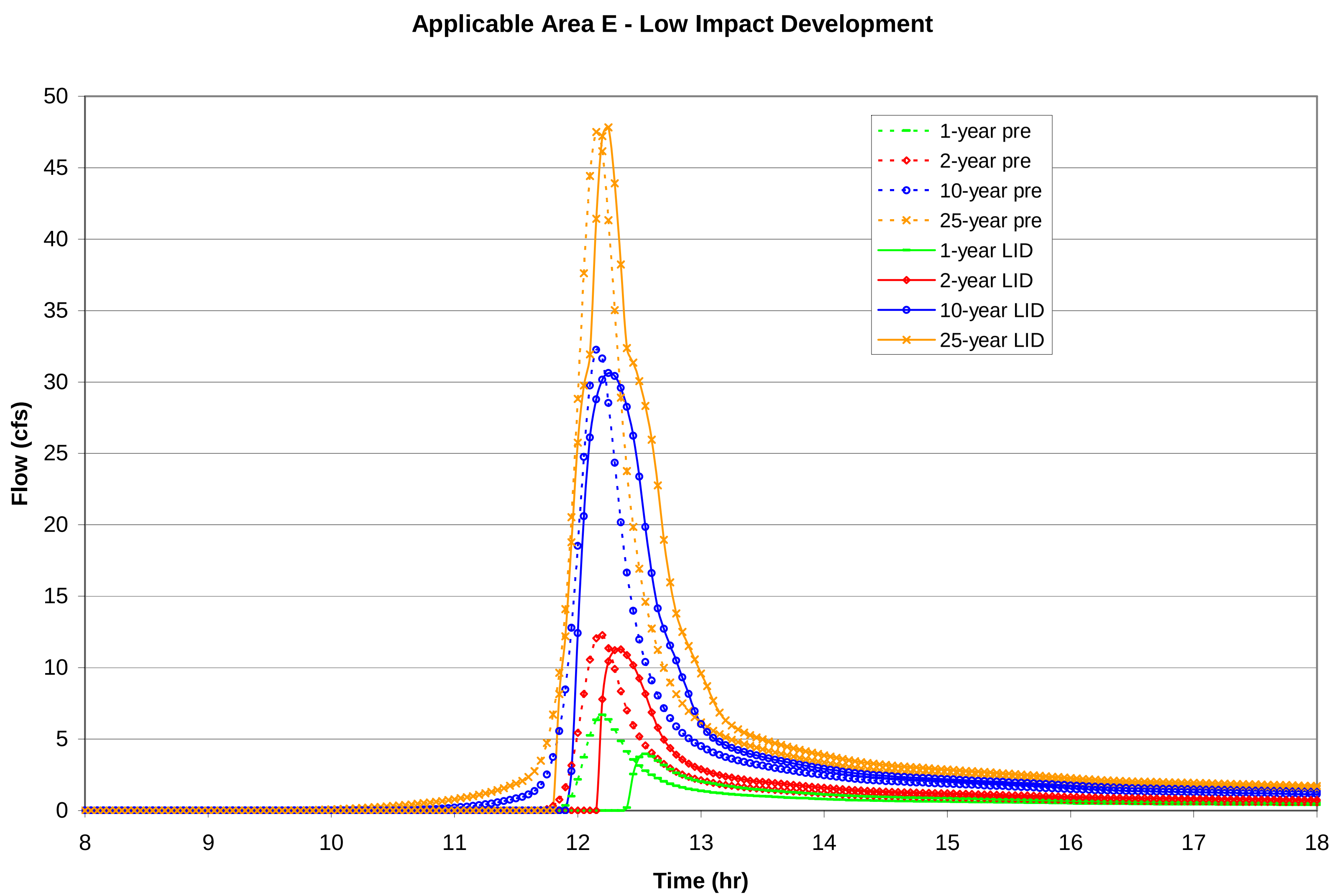
N-12. Pre- and post-development hydrographs for applicable area B with the Low Impact Development design.



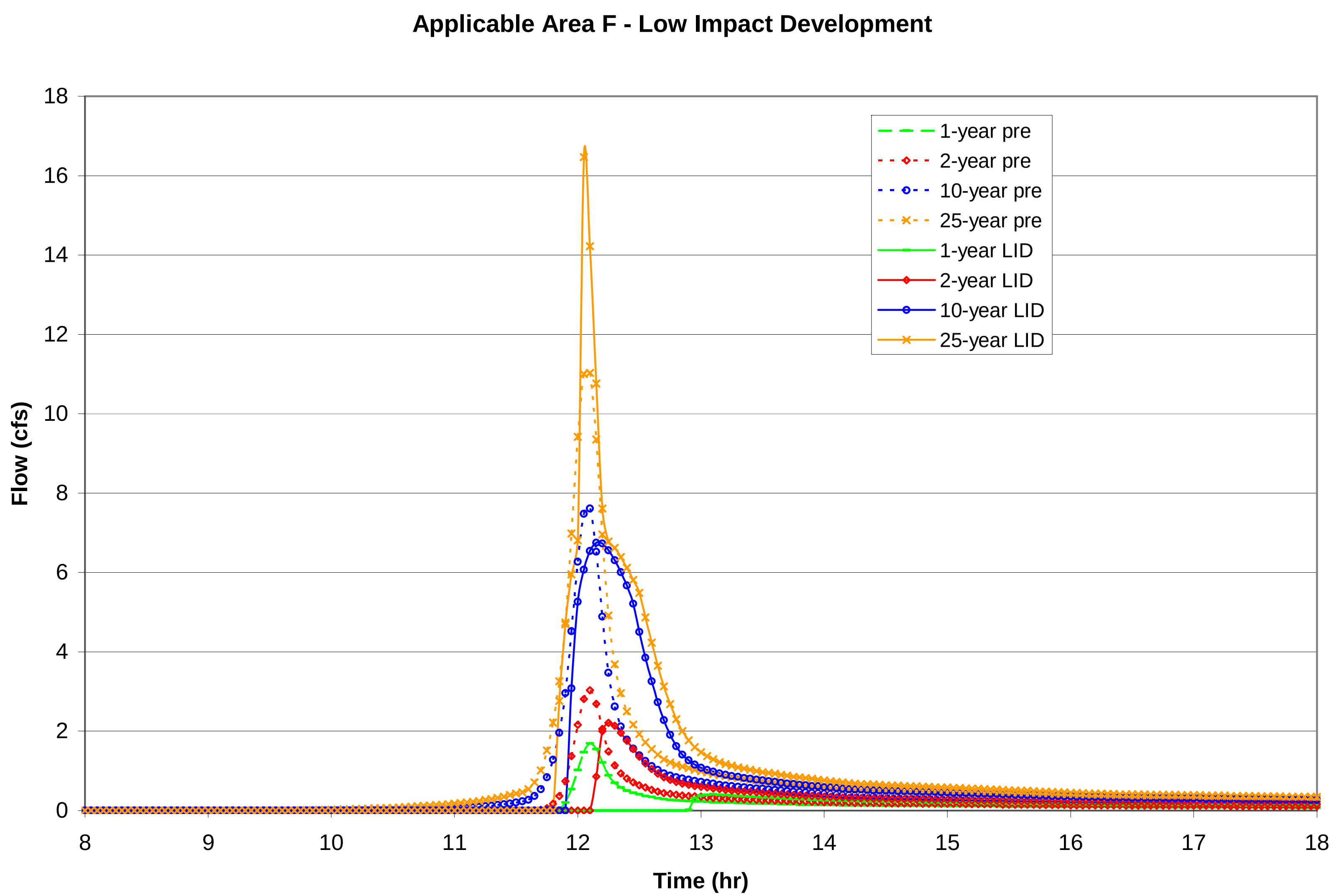
N-13. Pre- and post-development hydrographs for applicable area C with the Low Impact Development design.



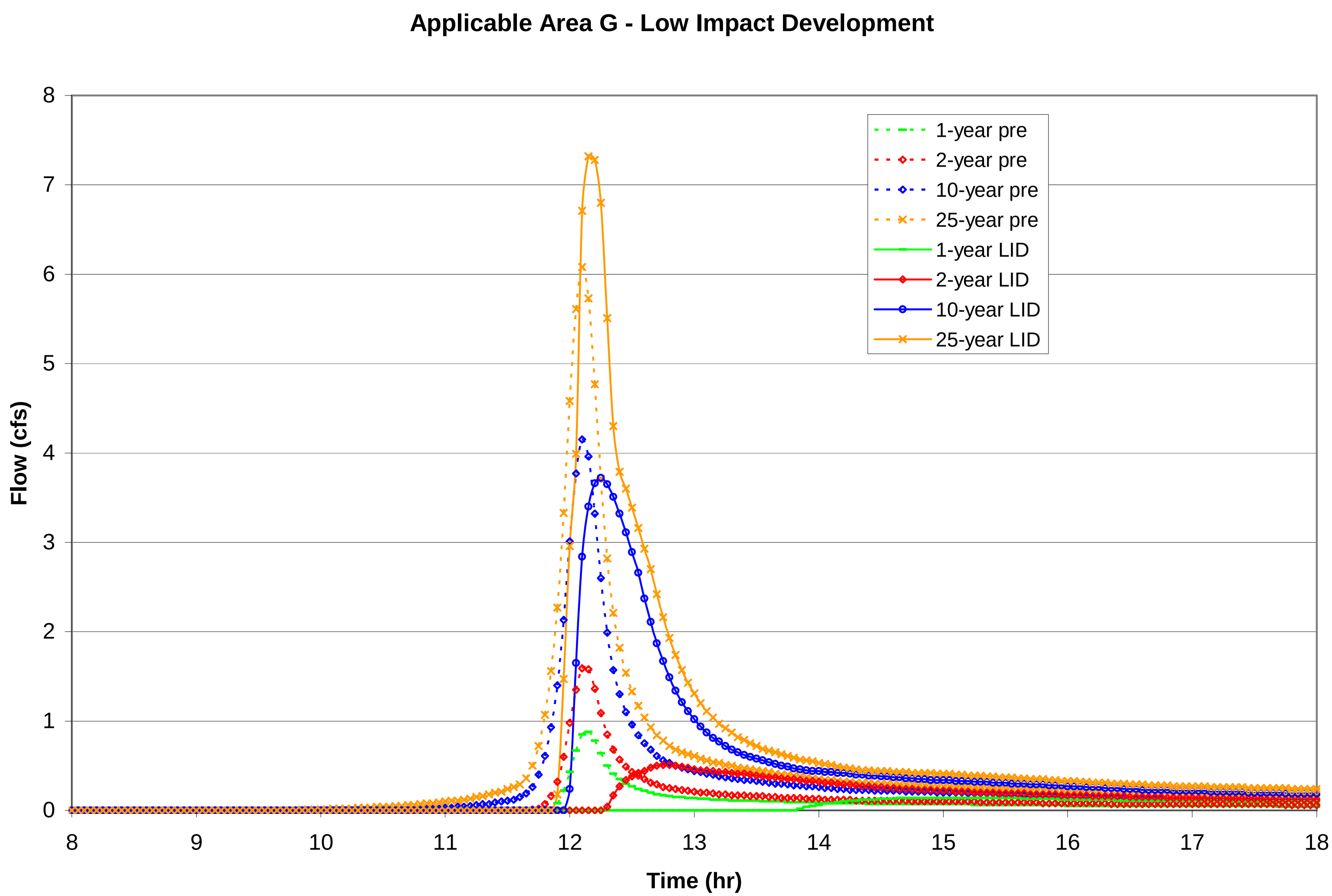
N-14. Pre- and post-development hydrographs for applicable area D with the Low Impact Development design.



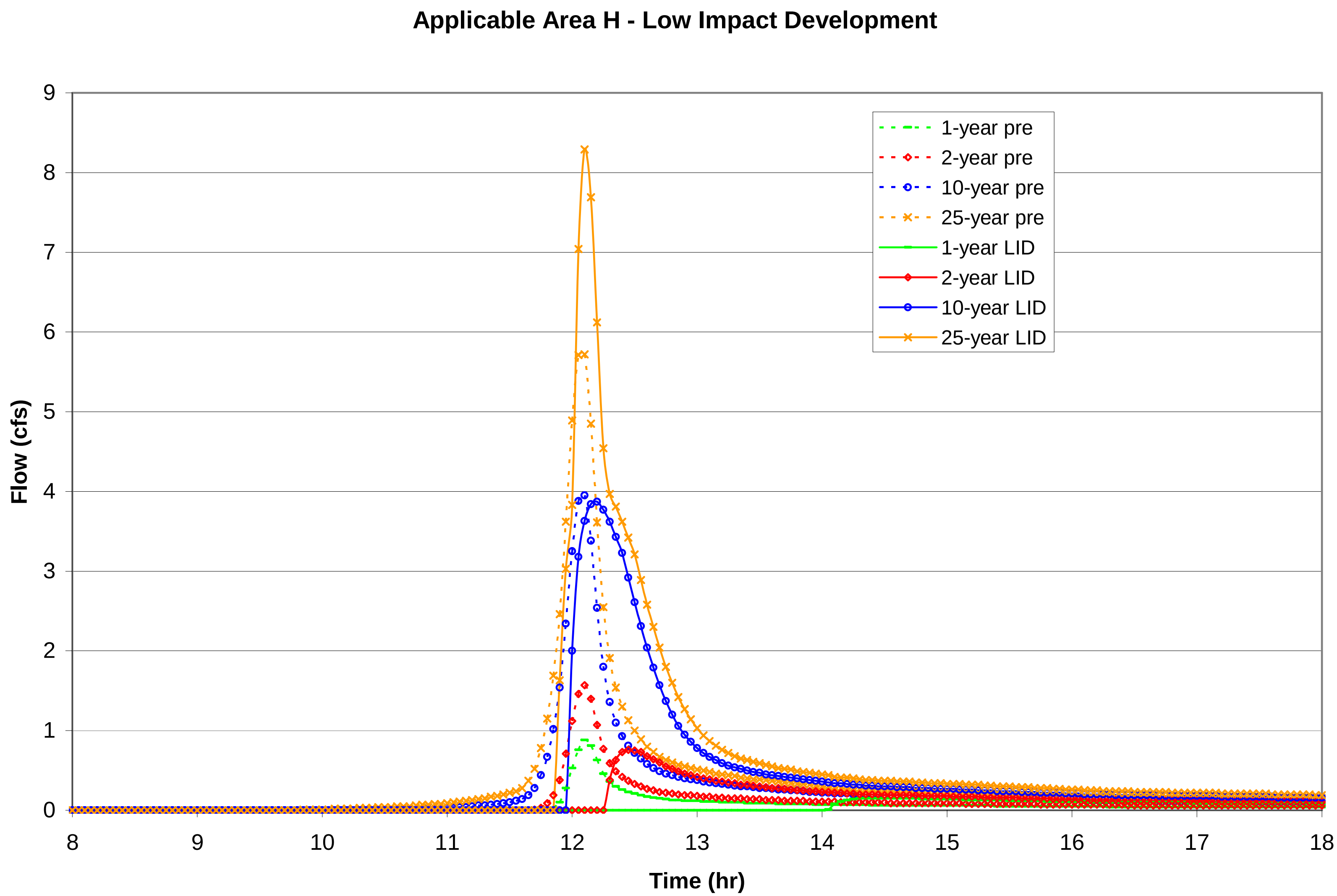
N-15. Pre- and post-development hydrographs for applicable area E with the Low Impact Development design.



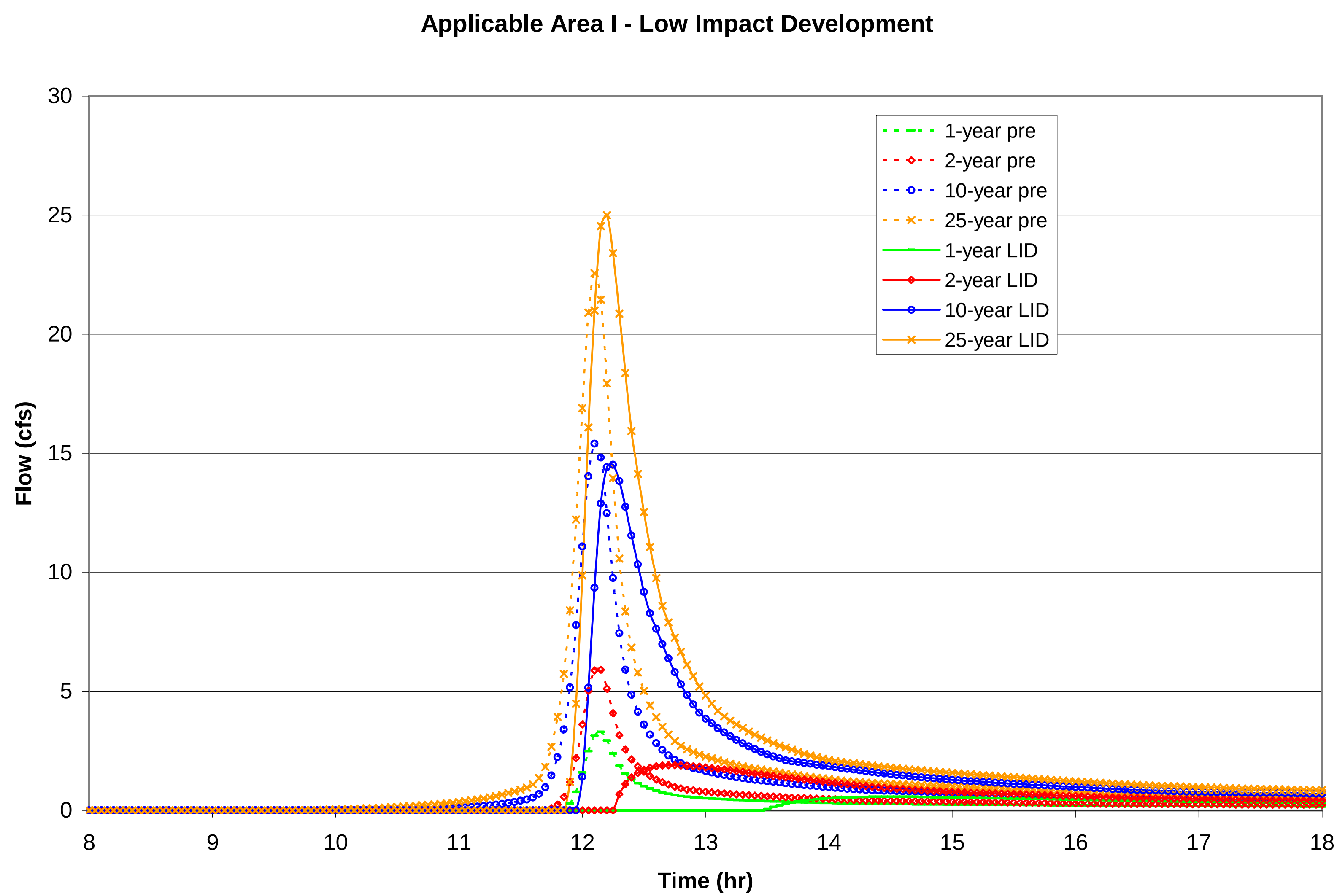
N-16. Pre- and post-development hydrographs for applicable area F with the Low Impact Development design.



N-17. Pre- and post-development hydrographs for applicable area G with the Low Impact Development design.



N-18. Pre- and post-development hydrographs for applicable area H with the Low Impact Development design.



N-19. Pre- and post-development hydrographs for applicable area I with the Low Impact Development design.

Appendix O: Information for the Full Spectrum Detention strategy

- **O-1: Corresponding elevation within each facility for the EURV calculation worksheet, plus 10%.**
- **O-2: Control Structure information for each facility serving each applicable area used to achieve the FSD design objectives.**
- **O-3: 72-hour drawdown for the EURV plus 10% for facility serving Applicable Area A.**
- **O-4: 72-hour drawdown for the EURV plus 10% for facility serving Applicable Area B.**
- **O-5: 72-hour drawdown for the EURV plus 10% for facility serving Applicable Area C.**
- **O-6: 72-hour drawdown for the EURV plus 10% for facility serving Applicable Area D.**
- **O-7: 72-hour drawdown for the EURV plus 10% for facility serving Applicable Area E.**
- **O-8: 72-hour drawdown for the EURV plus 10% for facility serving Applicable Area F.**
- **O-9: 72-hour drawdown for the EURV plus 10% for facility serving Applicable Area G.**
- **O-10: 72-hour drawdown for the EURV plus 10% for facility serving Applicable Area H.**
- **O-11: 72-hour drawdown for the EURV plus 10% for facility serving Applicable Area I.**
- **O-12: Post-development hydrographs for applicable area A with the Full Spectrum Detention design.**
- **O-13: Post-development hydrographs for applicable area B with the Full Spectrum Detention design.**
- **O-14: Post-development hydrographs for applicable area C with the Full Spectrum Detention design.**

- **O-15: Post-development hydrographs for applicable area D with the Full Spectrum Detention design.**
- **O-16: Post-development hydrographs for applicable area E with the Full Spectrum Detention design.**
- **O-17: Post-development hydrographs for applicable area F with the Full Spectrum Detention design.**
- **O-18: Post-development hydrographs for applicable area G with the Full Spectrum Detention design.**
- **O-19: Post-development hydrographs for applicable area H with the Full Spectrum Detention design.**
- **O-20: Post-development hydrographs for applicable area I with the Full Spectrum Detention design.**

O-1. Corresponding elevation within each facility for the EURV calculation worksheet, plus 10%.

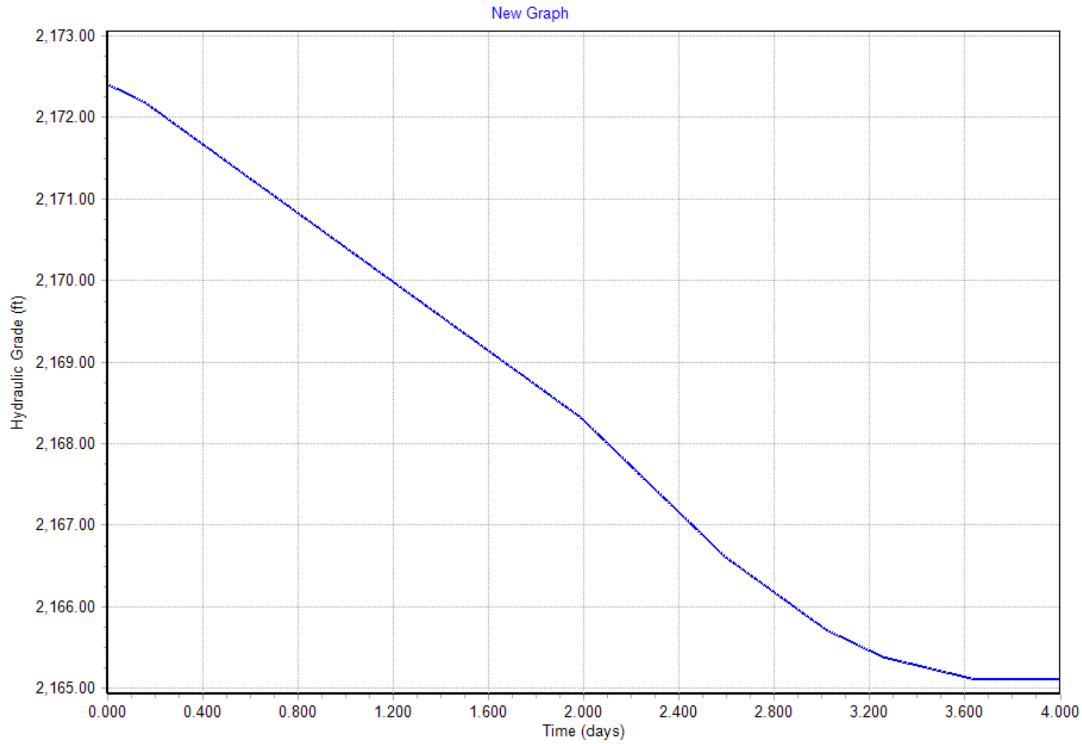
Applicable Area	% Impervious (x)	EURV as inches per acre (0.0235x)	Area (acres)	EURV (ft3)	EURV + 10% (ft3)	Corresponding Stage in Facility (ft)
A	34.93	0.82	30.27	90,199	99,219	2172.4
B	39.83	0.94	5.97	20,283	22,312	2,244.2
C	46.05	1.08	7.56	29,695	32,665	2,210.2
D	21.42	0.50	6.05	11,058	12,164	2163.8
E	15.51	0.36	21.53	28,484	31,333	2145.3
F	4.65	0.11	25.51	10,122	11,134	2126.2
G	58.66	1.38	2.35	11,760	12,935	2114.2
H	73.41	1.73	2.07	12,938	14,231	2102.8
I	39.13	0.92	8.77	29,282	32,210	2081.9

O-2. Control Structure information for each facility serving each applicable area used to achieve the FSD design objectives.

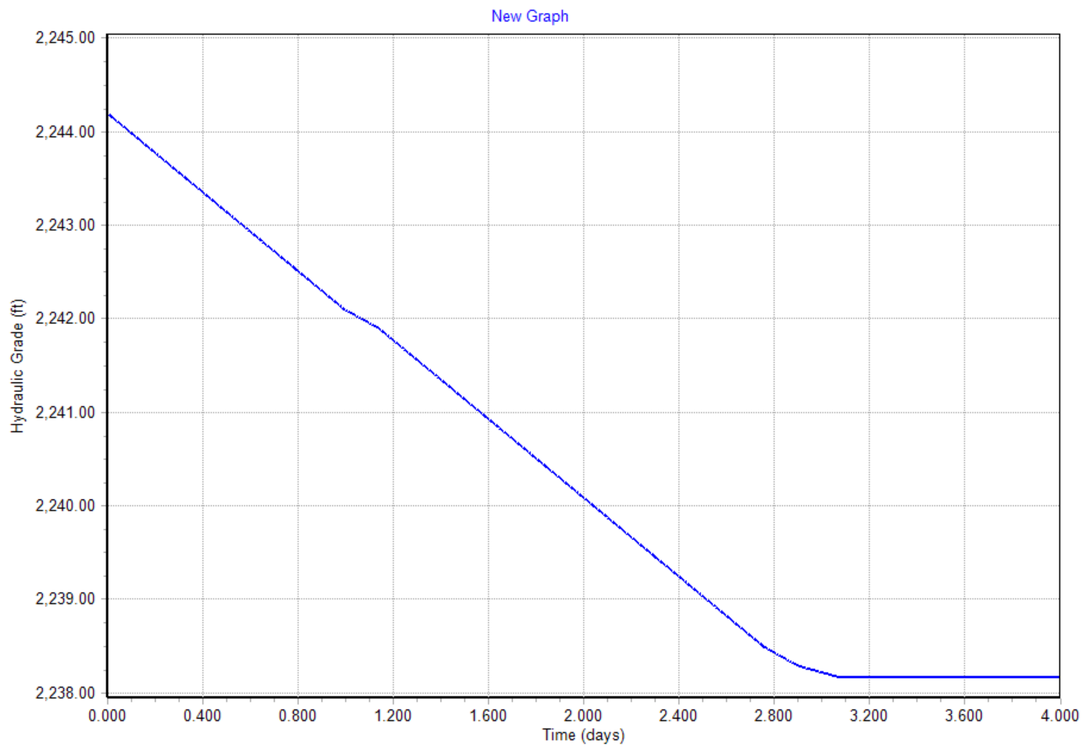
Facility Serving Applicable Area	EURV + 10% Drawdown Orifice		Secondary Orifice		Riser	
	Dia. (in)	Invert (ft)	Dia. (in)	Invert (ft)	Dia. (in)	Crest (ft)
A	2.7	2,165.00	N/A		60	2,172.40
B	1.4	2,238.10	N/A		36	2,244.20
C	1.8	2,204.00	N/A		27	2,210.20
D	1.2	2,160.00	N/A		20	2,163.80
E	1.4	2,137.26	19 (3)	2,145.30	N/A	
F	1.0	2,121.30	19	2,126.20	N/A	
G	1.2	2,110.50	60	2,114.20	N/A	
H	1.0	2,094.20	48	2,102.80	N/A	
I	2.0	2,078.50	N/A		28	2,081.90

*** Orifice coefficient = 0.6 in all cases**

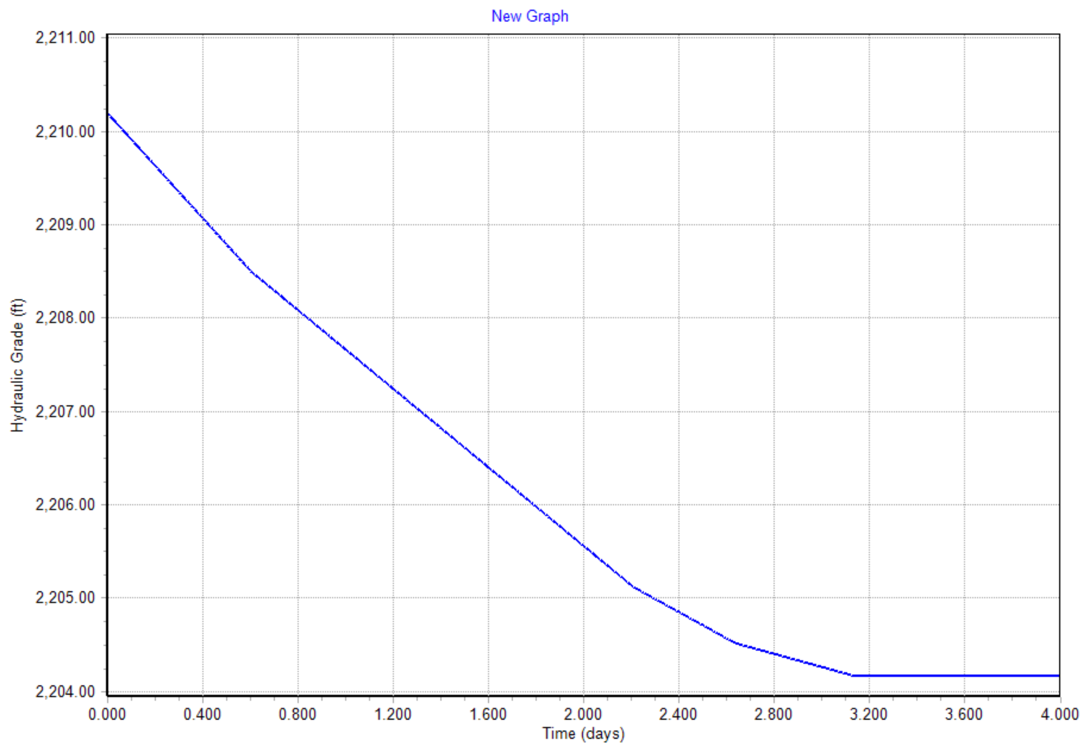
**** Weir coefficient = 3.1 in all cases; weir is suppressed type in all cases**



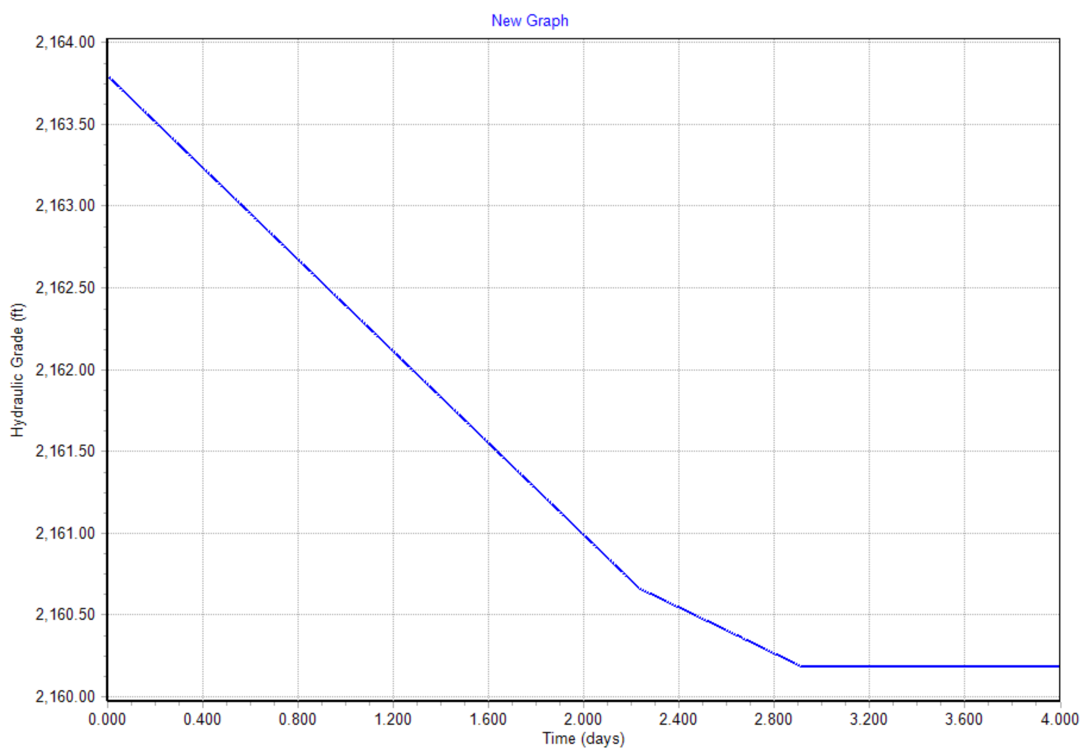
O-3. 72-hour drawdown for the EURV plus 10% for facility serving Applicable Area A. Drawdown is achieved with 2.7" orifice set at the invert of the facility.



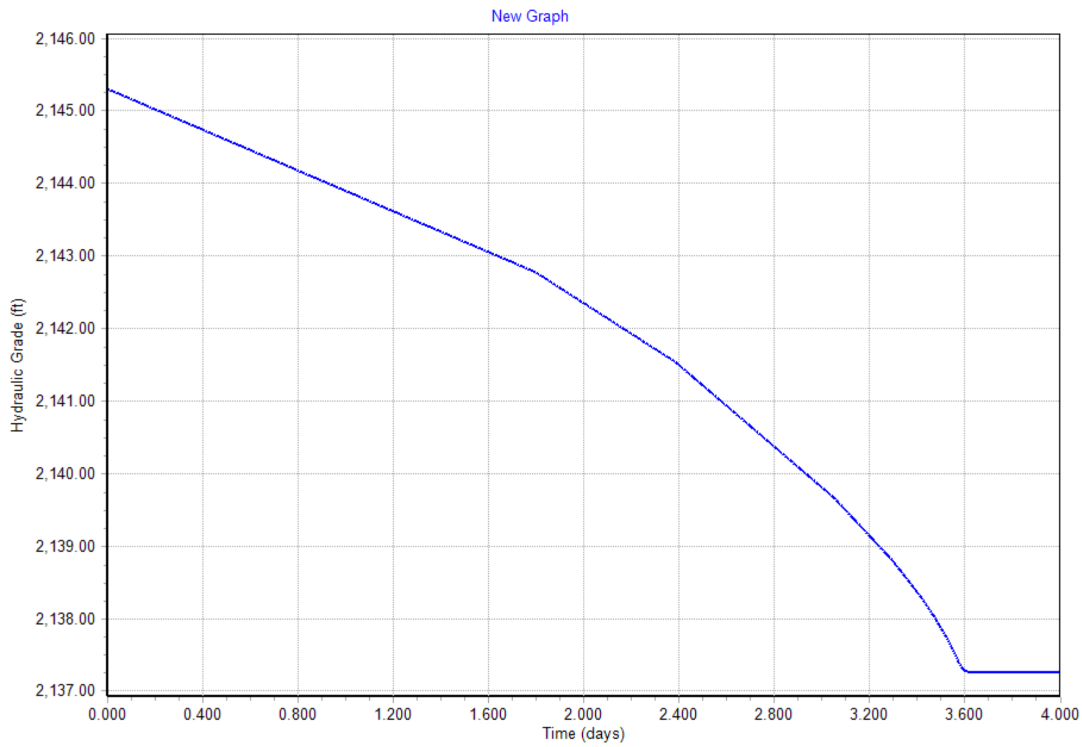
O-4. 72-hour drawdown for the EURV plus 10% for facility serving Applicable Area B. Drawdown is achieved with 1.4" orifice set at the invert of the facility.



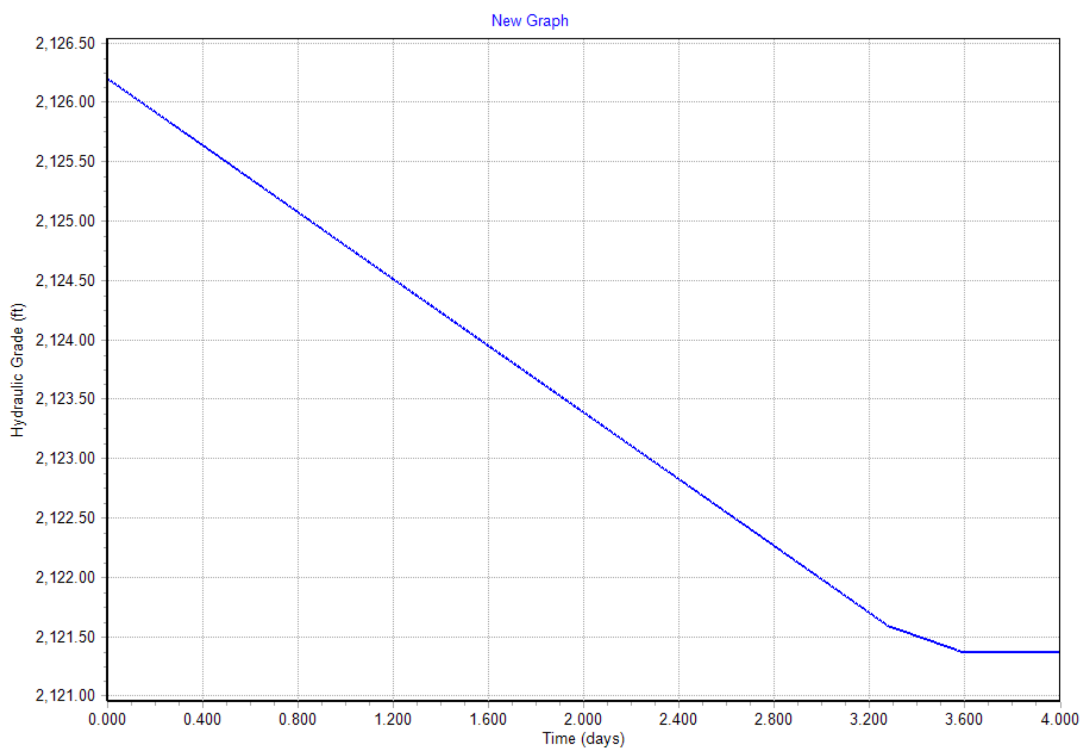
O-5. 72-hour drawdown for the EURV plus 10% for facility serving Applicable Area C. Drawdown is achieved with 1.8" orifice set at the invert of the facility.



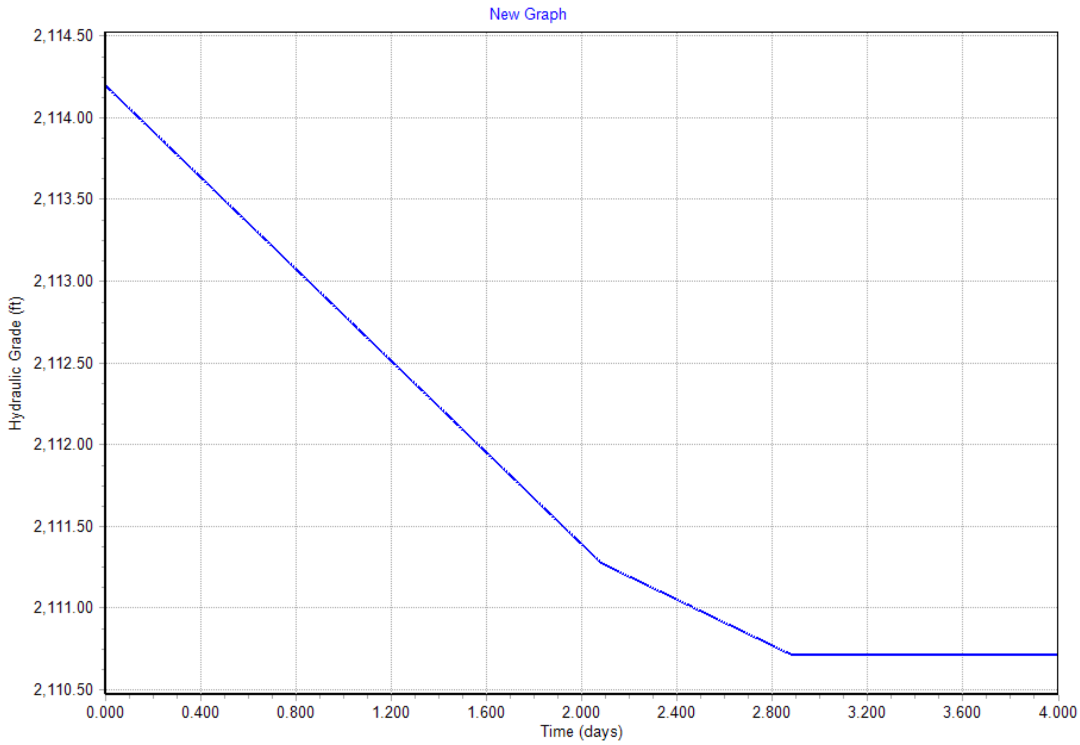
O-6. 72-hour drawdown for the EURV plus 10% for facility serving Applicable Area D. Drawdown is achieved with 1.2" orifice set at the invert of the facility.



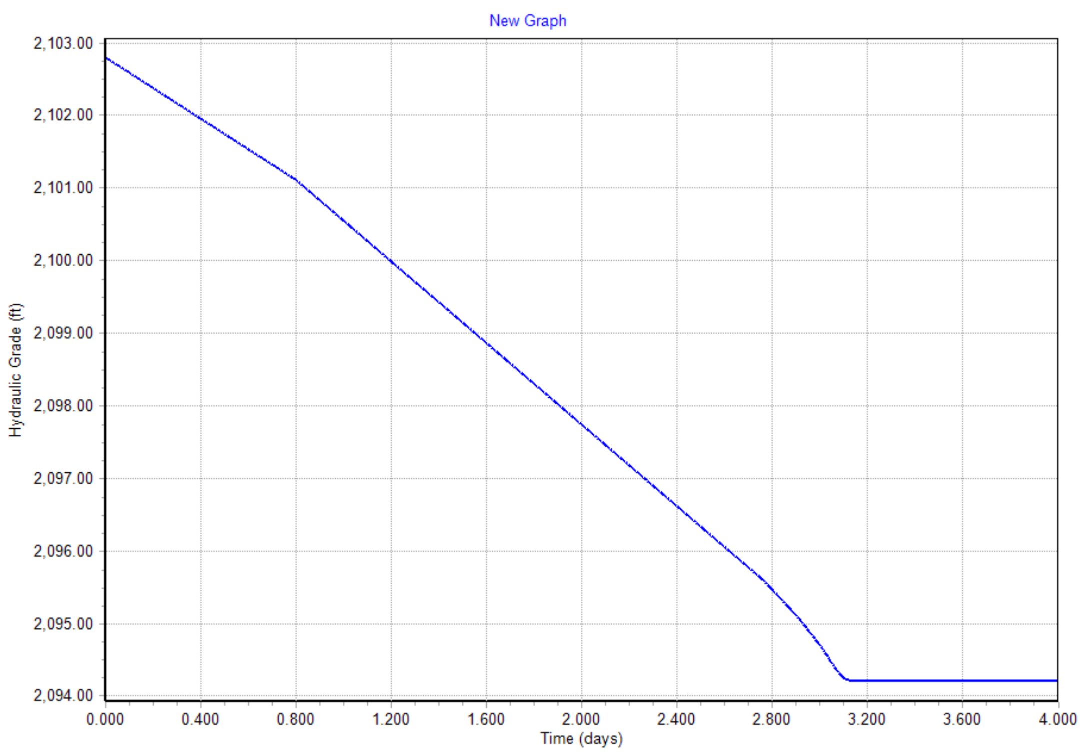
O-7. 72-hour drawdown for the EURV plus 10% for facility serving Applicable Area E. Drawdown is achieved with 1.4" orifice set at the invert of the facility.



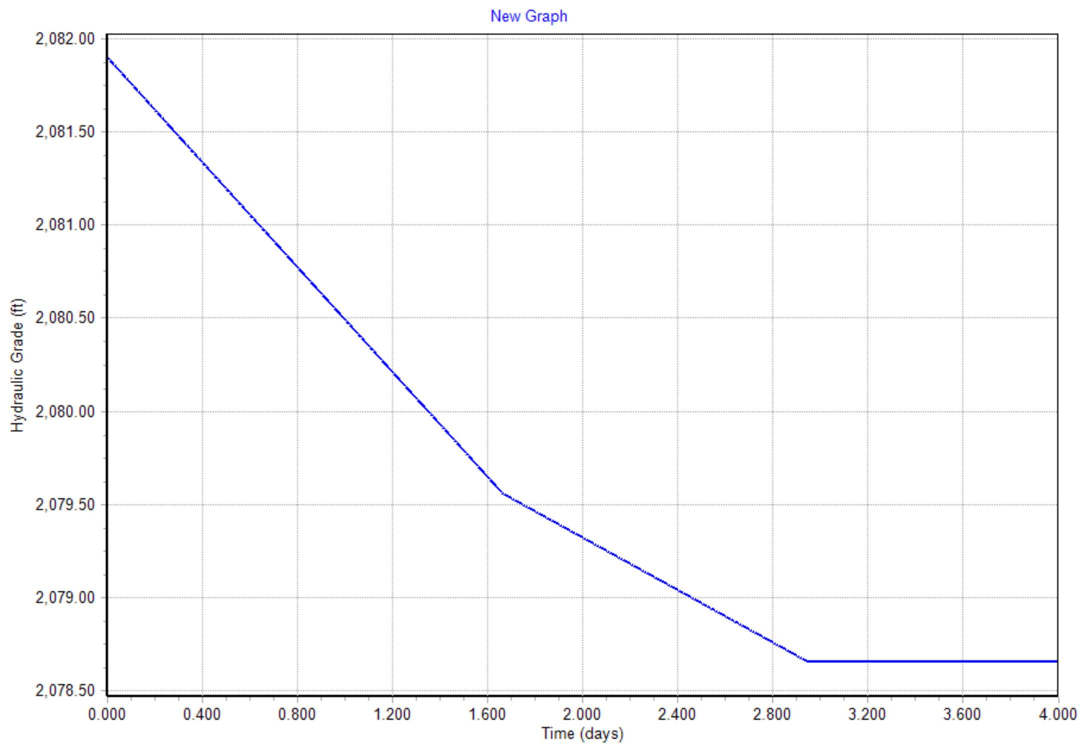
O-8. 72-hour drawdown for the EURV plus 10% for facility serving Applicable Area F. Drawdown is achieved with 1.0" orifice set at the invert of the facility.



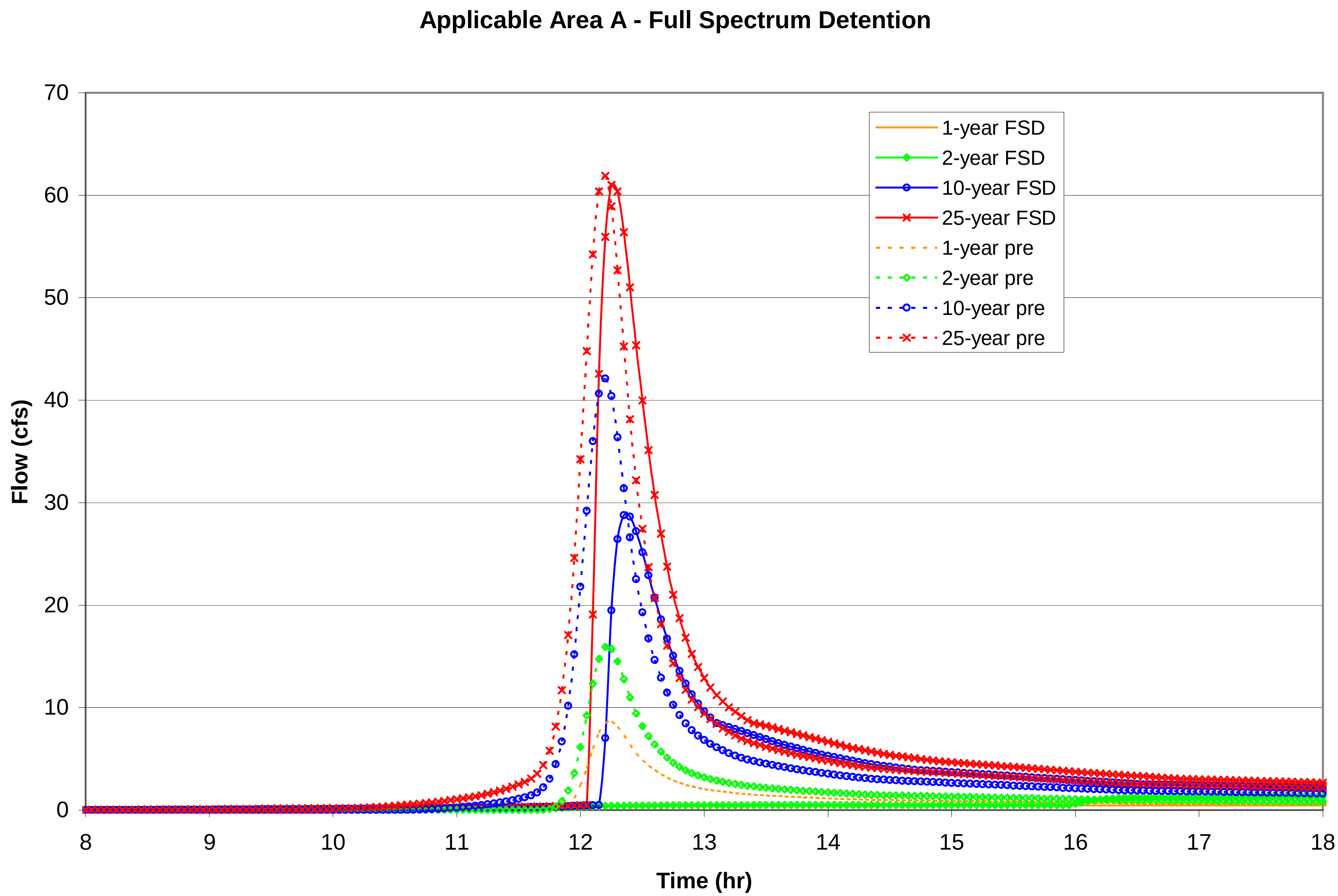
O-9. 72-hour drawdown for the EURV plus 10% for facility serving Applicable Area G. Drawdown is achieved with 1.2" orifice set at the invert of the facility.



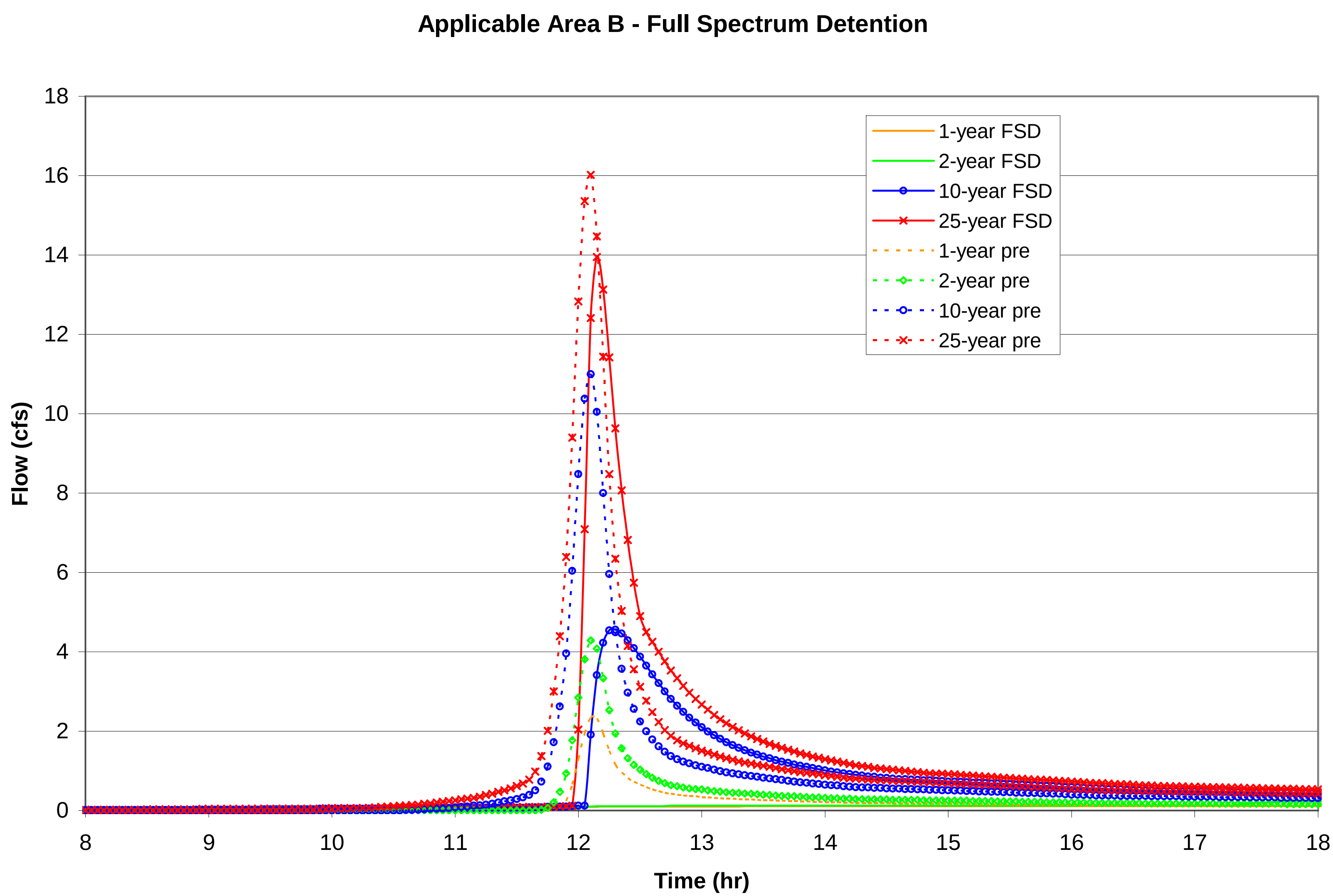
O-10. 72-hour drawdown for the EURV plus 10% for facility serving Applicable Area H. Drawdown is achieved with 1.0" orifice set at the invert of the facility.



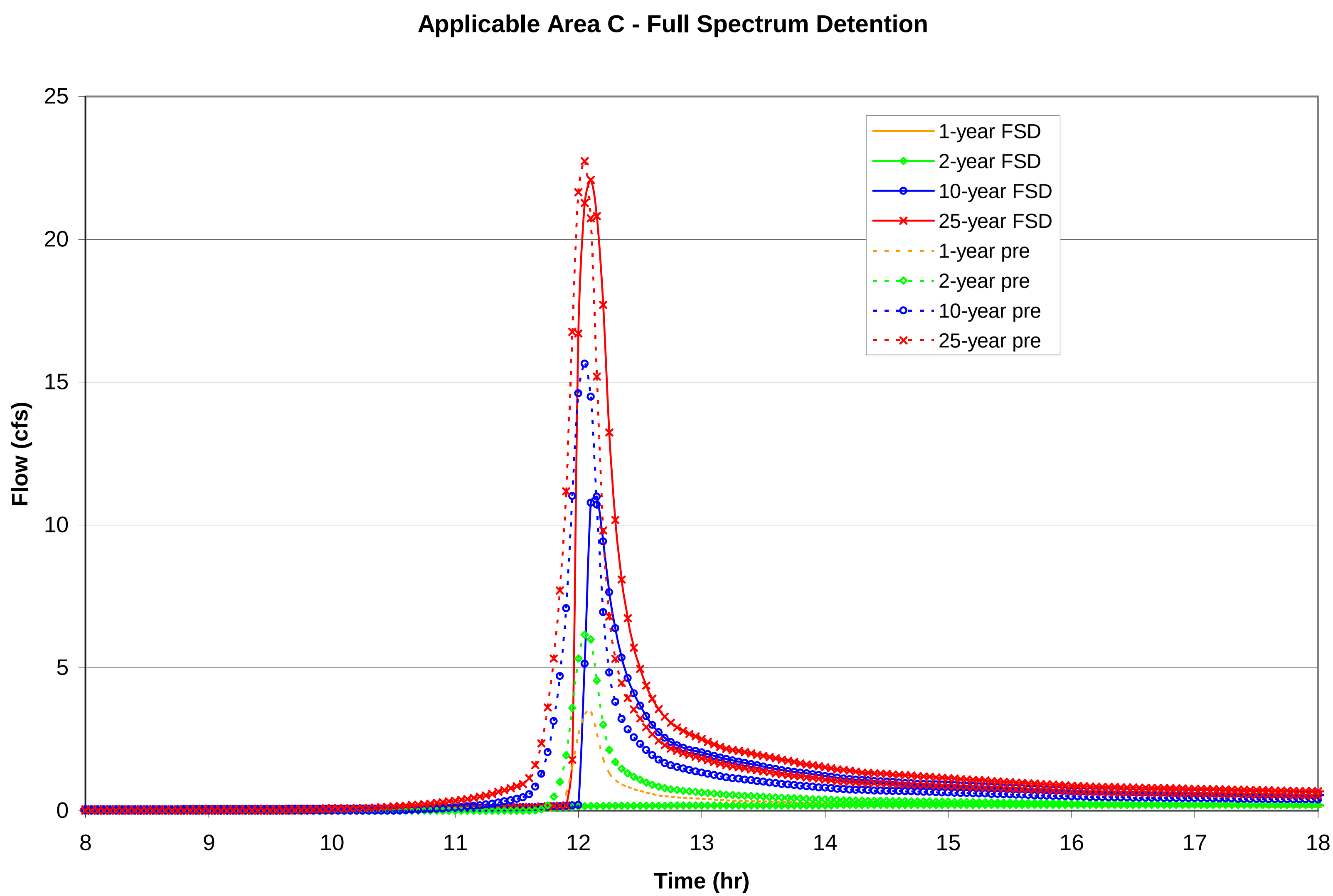
O-11. 72-hour drawdown for the EURV plus 10% for facility serving Applicable Area I. Drawdown is achieved with 2.0" orifice set at the invert of the facility.



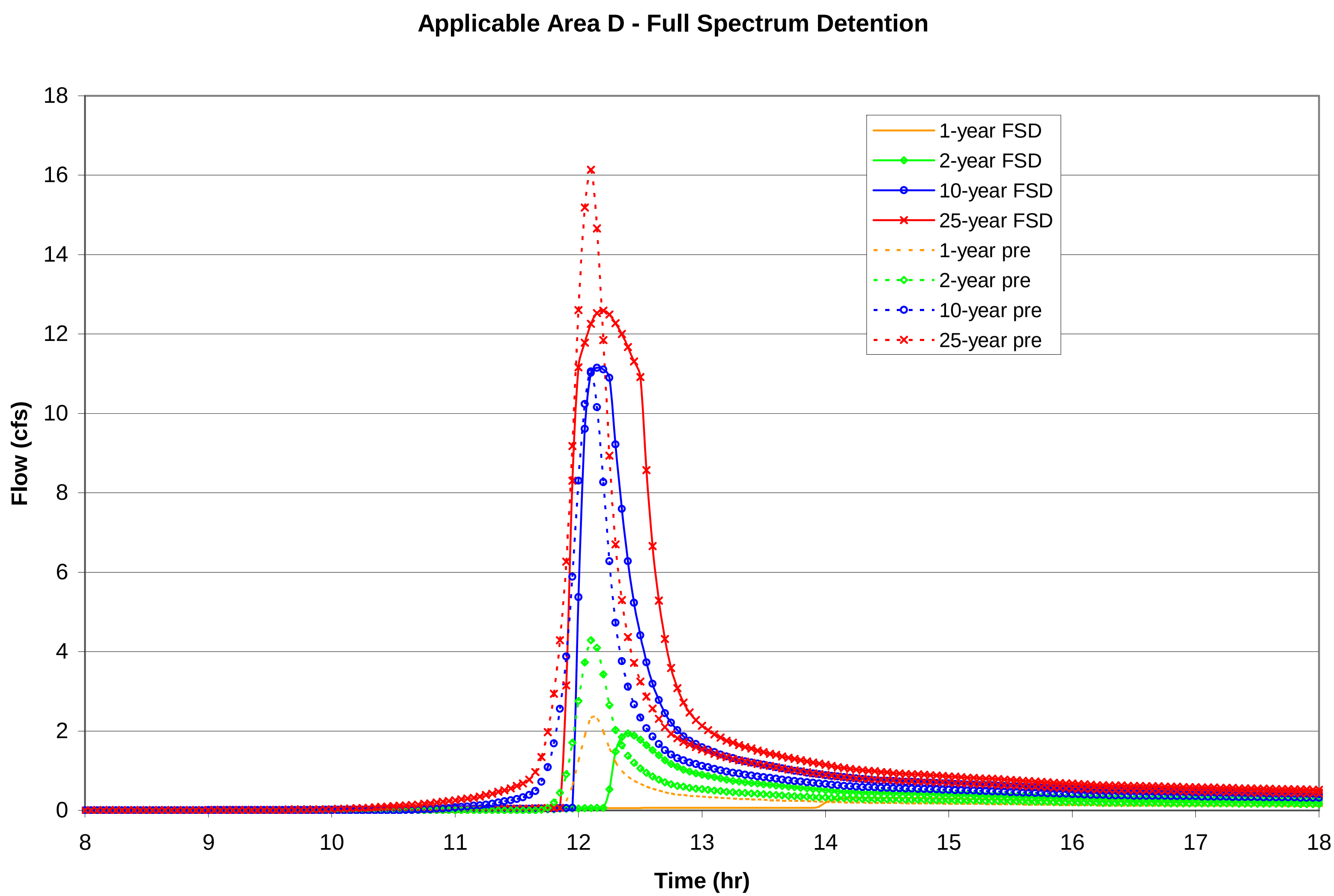
O-12. Post-development hydrographs for applicable area A with the Full Spectrum Detention design.



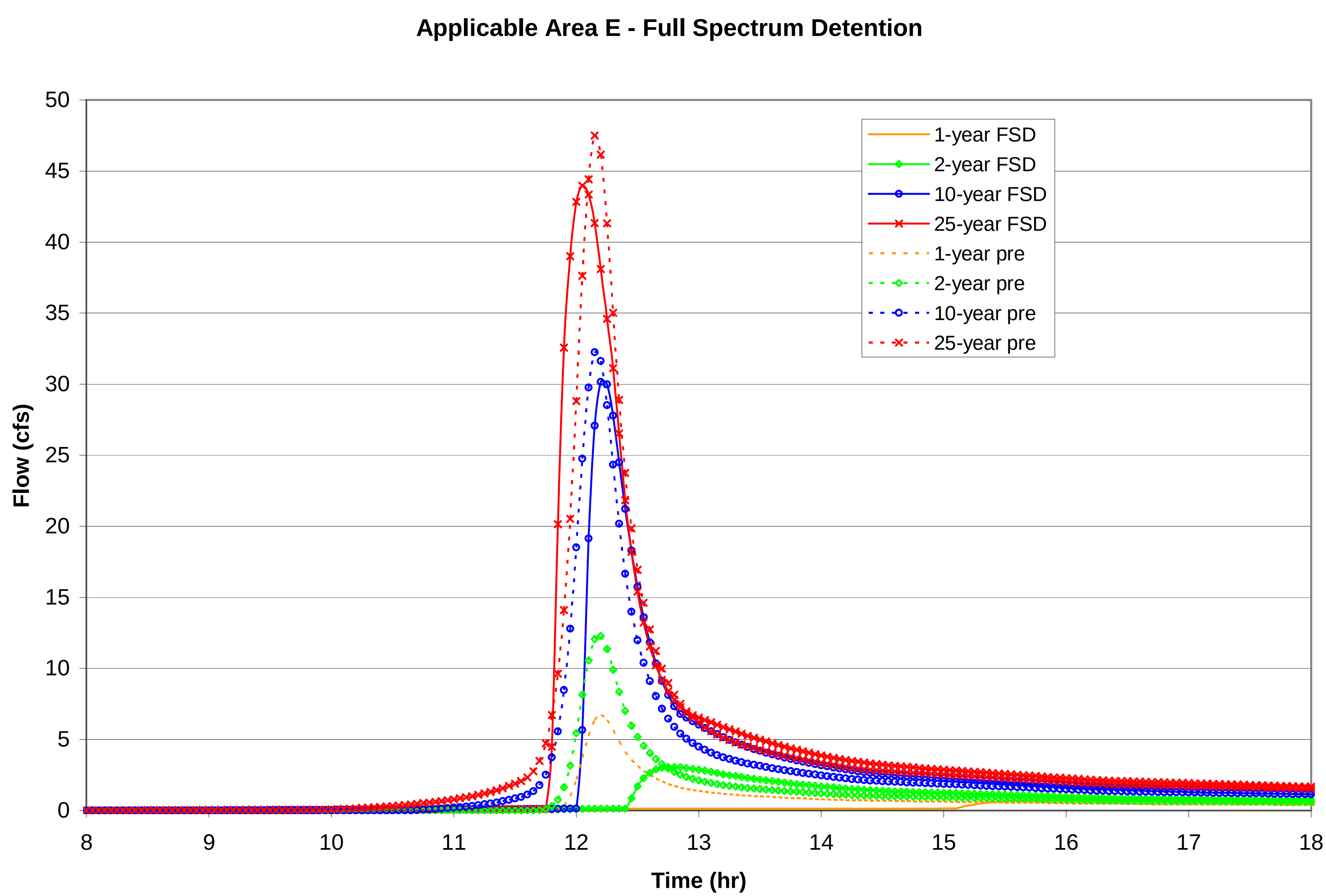
O-13. Post-development hydrographs for applicable area B with the Full Spectrum Detention design.



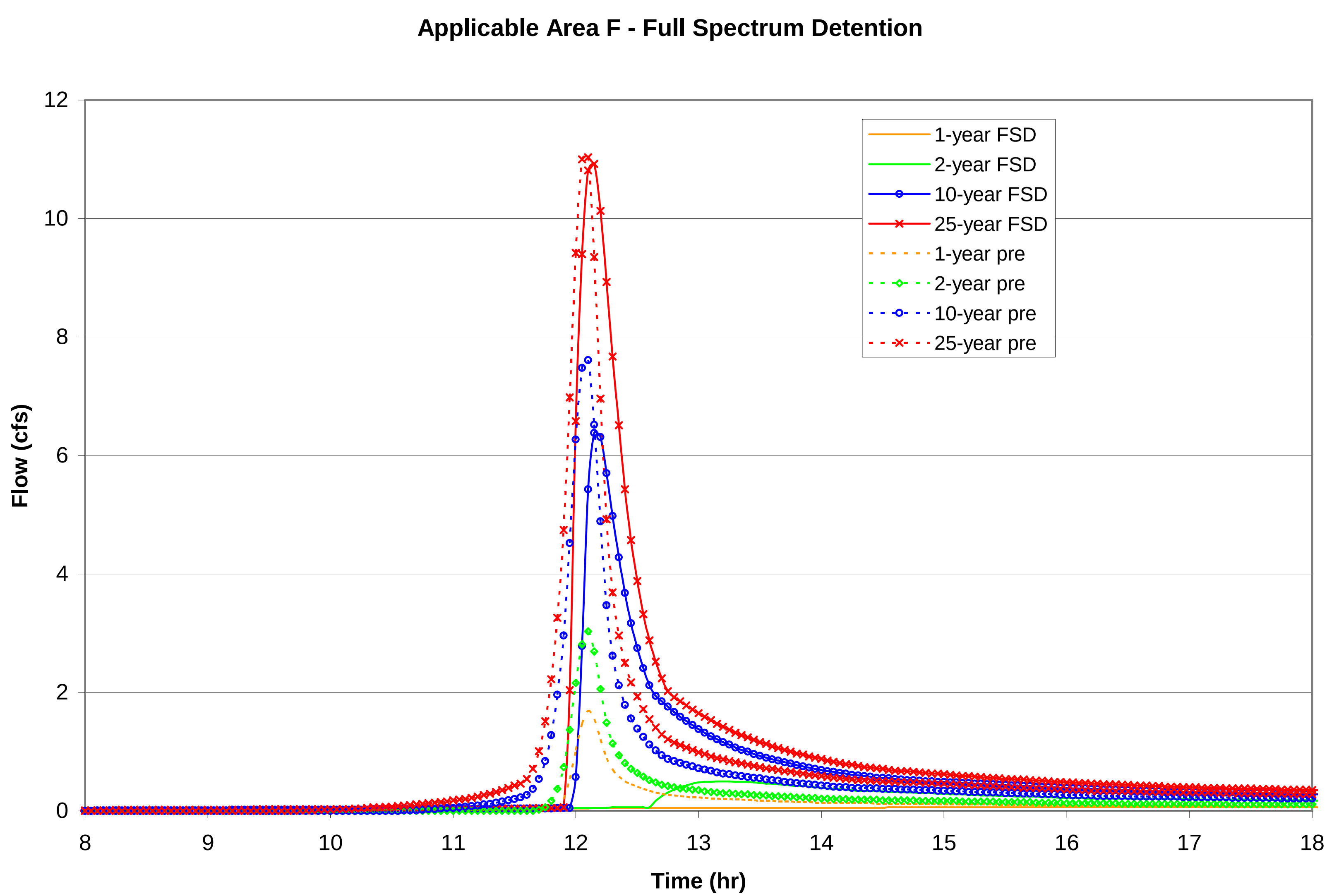
O-14. Post-development hydrographs for applicable area C with the Full Spectrum Detention design.



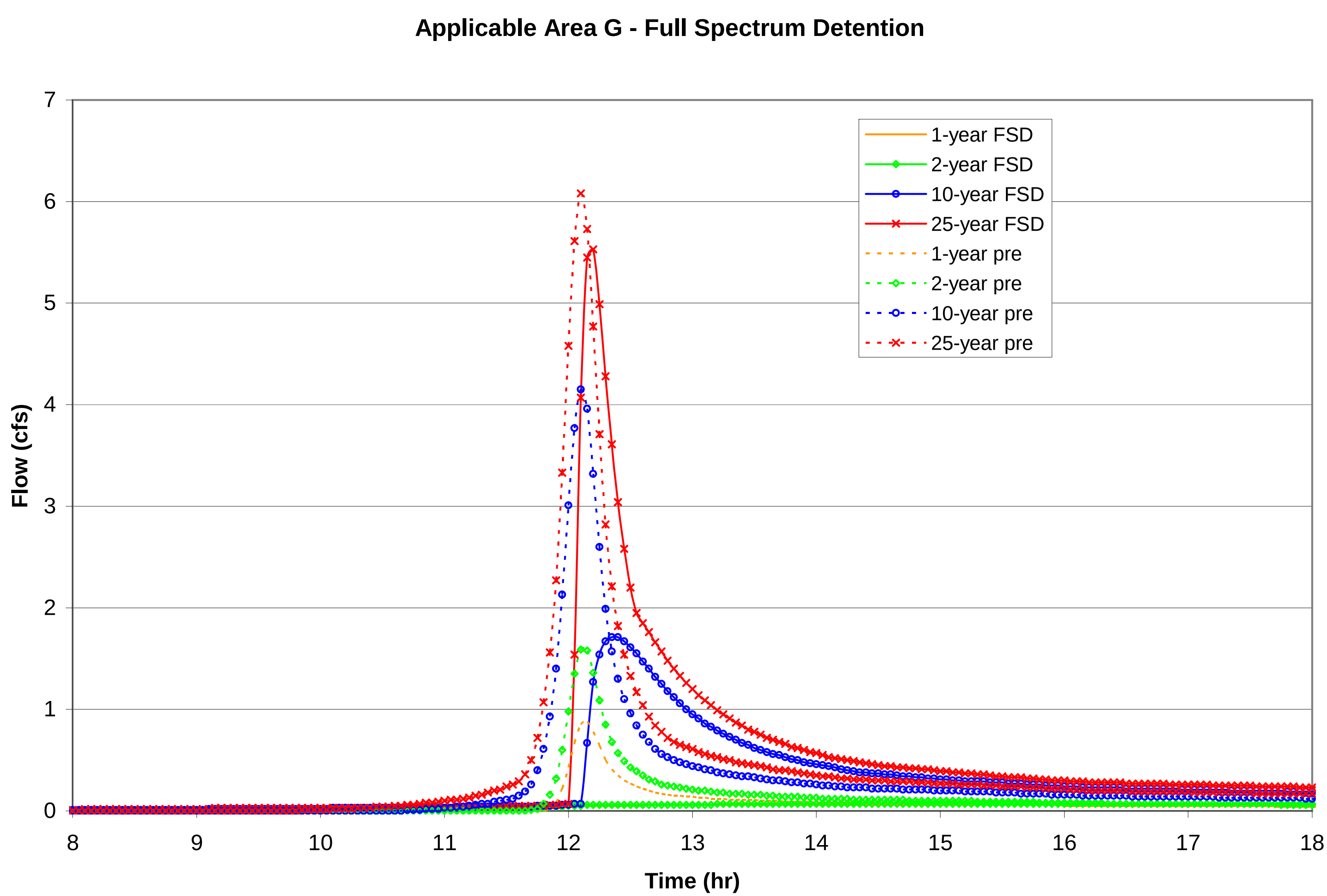
O-15. Post-development hydrographs for applicable area D with the Full Spectrum Detention design.



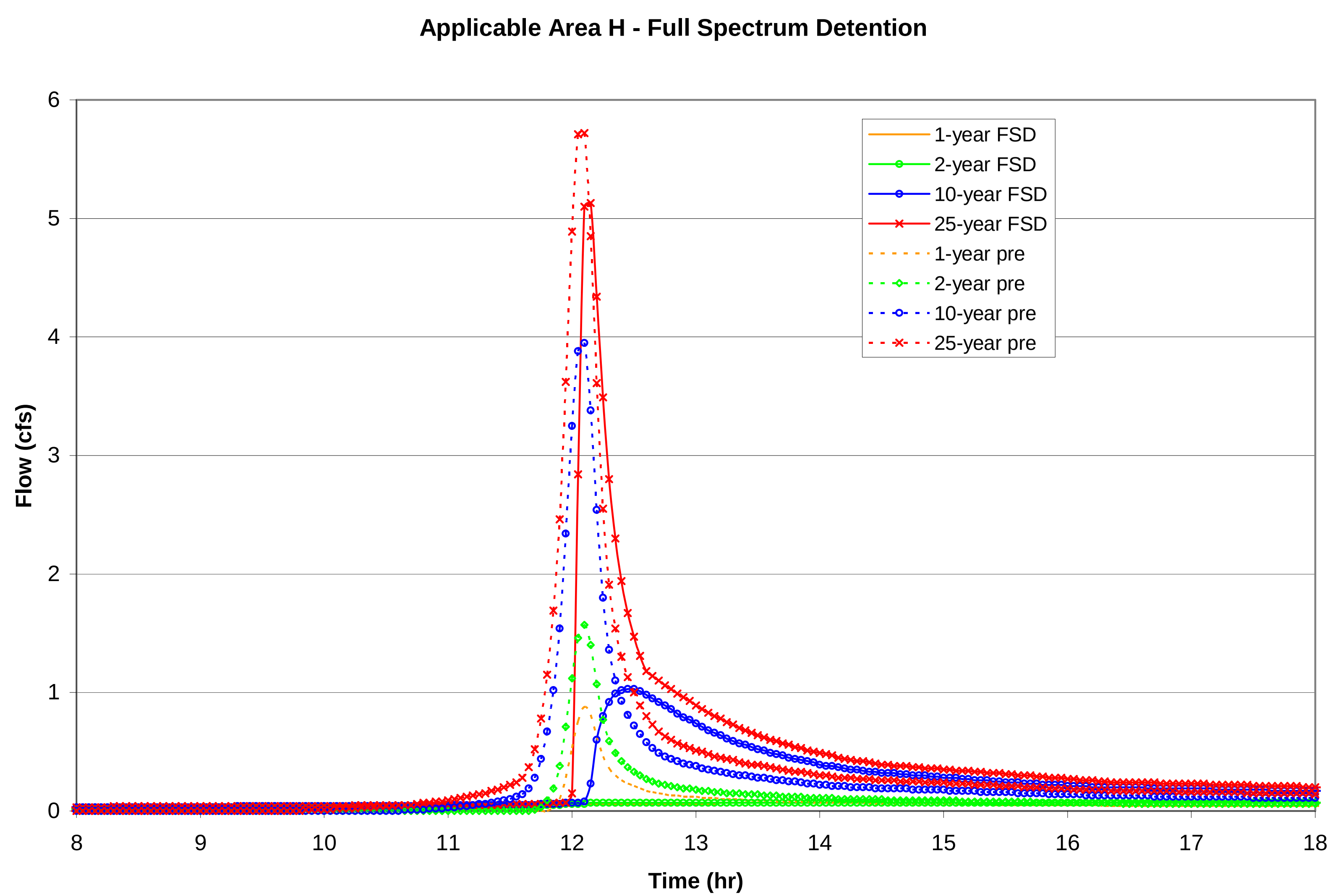
O-16. Post-development hydrographs for applicable area E with the Full Spectrum Detention design.



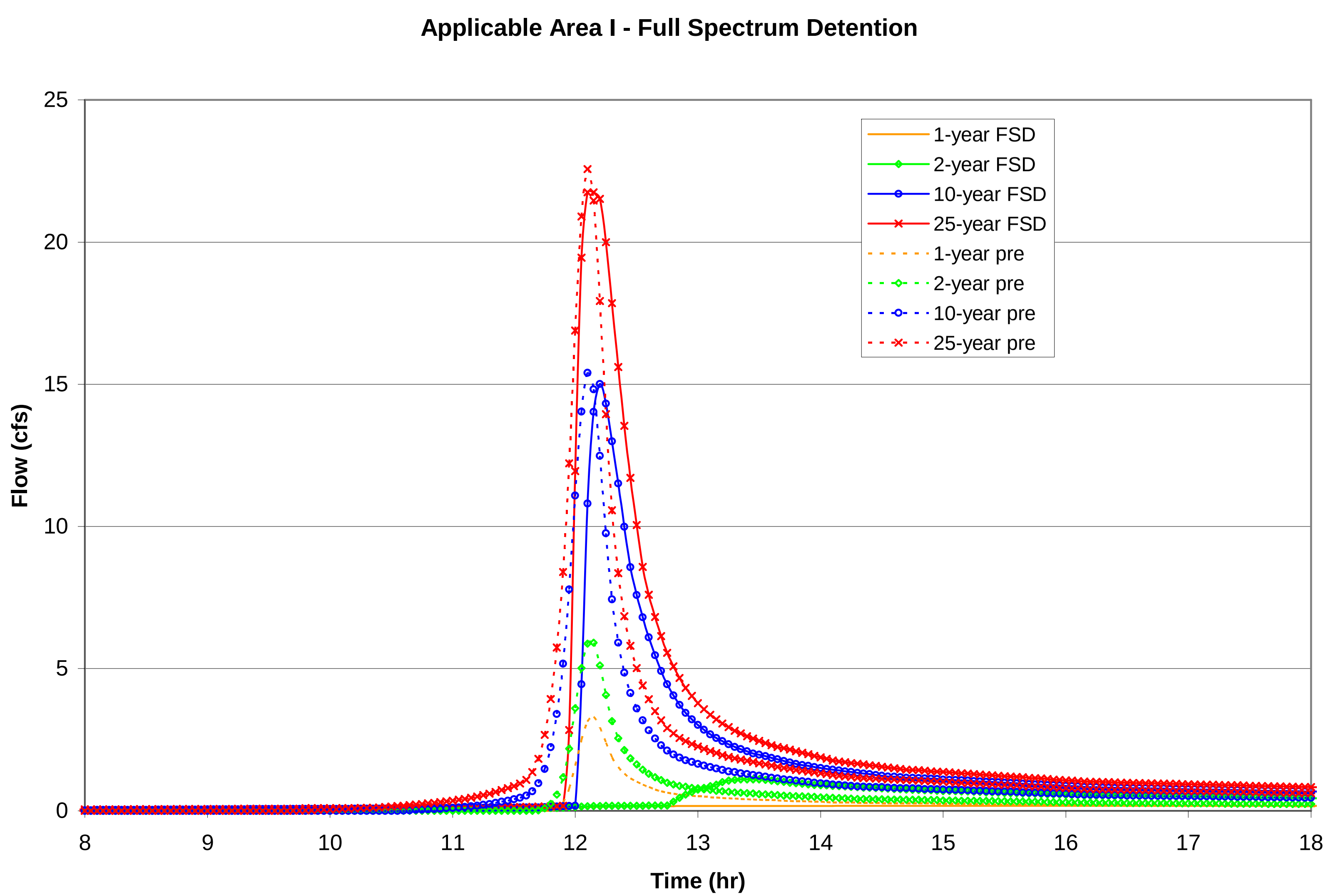
O-17. Post-development hydrographs for applicable area F with the Full Spectrum Detention design.



O-18. Post-development hydrographs for applicable area G with the Full Spectrum Detention design.



O-19. Post-development hydrographs for applicable area H with the Full Spectrum Detention design.



O-20. Post-development hydrographs for applicable area I with the Full Spectrum Detention design.

Appendix P: Information for the Volumetric Design Procedure strategy

- **P-1: Control Structure information for each facility serving each applicable area used to achieve the VDP design objectives.**
- **P-2: Comparison of pre- and post development volumes released during the critical time period for the 2- and 25-year design storms.**
- **P-3: Post-development hydrographs for applicable area A with the Volumetric Design Procedure design.**
- **P-4: Post-development hydrographs for applicable area B with the Volumetric Design Procedure design.**
- **P-5: Post-development hydrographs for applicable area C with the Volumetric Design Procedure design.**
- **P-6: Post-development hydrographs for applicable area D with the Volumetric Design Procedure design.**
- **P-7: Post-development hydrographs for applicable area E with the Volumetric Design Procedure design.**
- **P-8: Post-development hydrographs for applicable area F with the Volumetric Design Procedure design.**
- **P-9: Post-development hydrographs for applicable area G with the Volumetric Design Procedure design.**
- **P-10: Post-development hydrographs for applicable area H with the Volumetric Design Procedure design.**
- **P-11: Post-development hydrographs for applicable area I with the Volumetric Design Procedure design.**

P-1. Control Structure information for each facility serving each applicable area used to achieve the VDP design objectives.

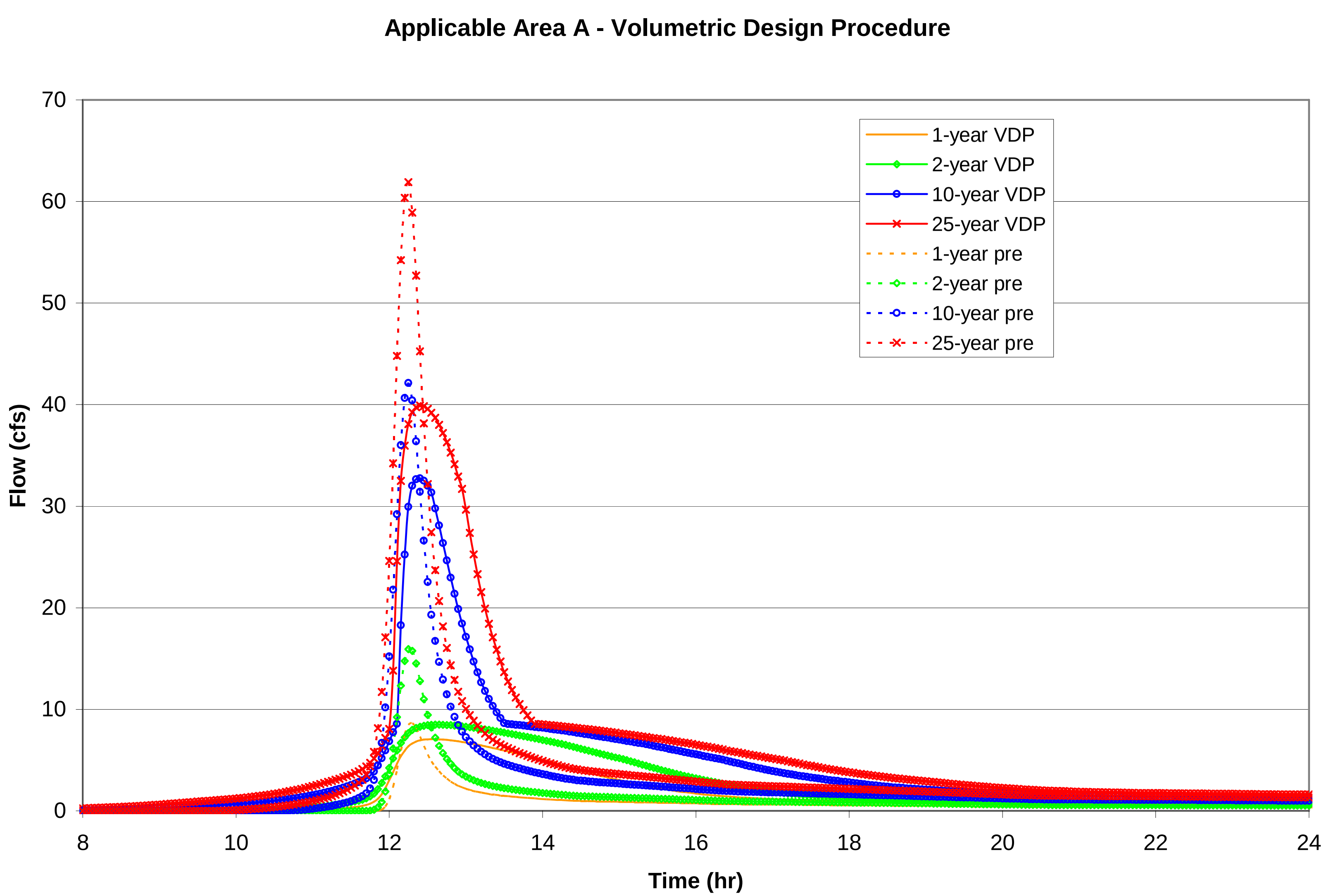
Facility Serving Applicable Area	Primary Orifice*		Riser**	
	Dia (in)	Invert (ft)	Dia (in)	Crest (ft)
A	13.0	2,165.00	26.0	2,170.00
B	6.0	2,238.10	12.0	2,242.10
C	7.0	2,204.00	12.0	2,207.80
D	8.0	2,160.00	12.0	2,163.00
E	10.0	2,137.26	21.0	2,143.70
F	5.0	2,121.30	10.0	2,125.10
G	3.5	2,110.50	10.0	2,113.10
H	1.5	2,094.20	10.0	2,101.30
I	8.0	2,078.50	18.0	2,081.30

* Orifice coefficient = 0.6 in all cases

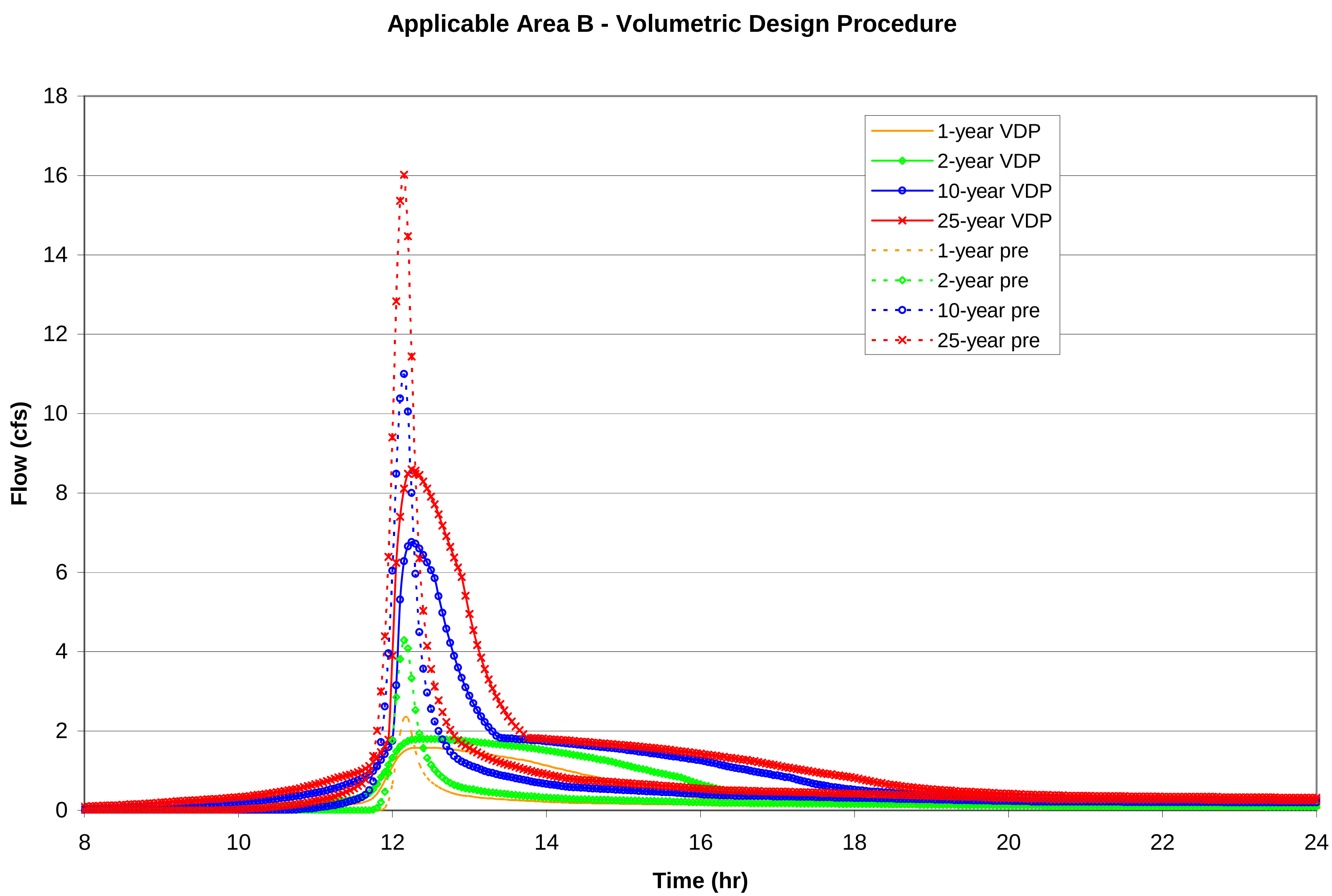
** Weir coefficient = 3.1 in all cases; Weir is suppressed type in all cases

P-2. Comparison of pre- and post development volumes released during the critical time period for the 2- and 25-year design storms.

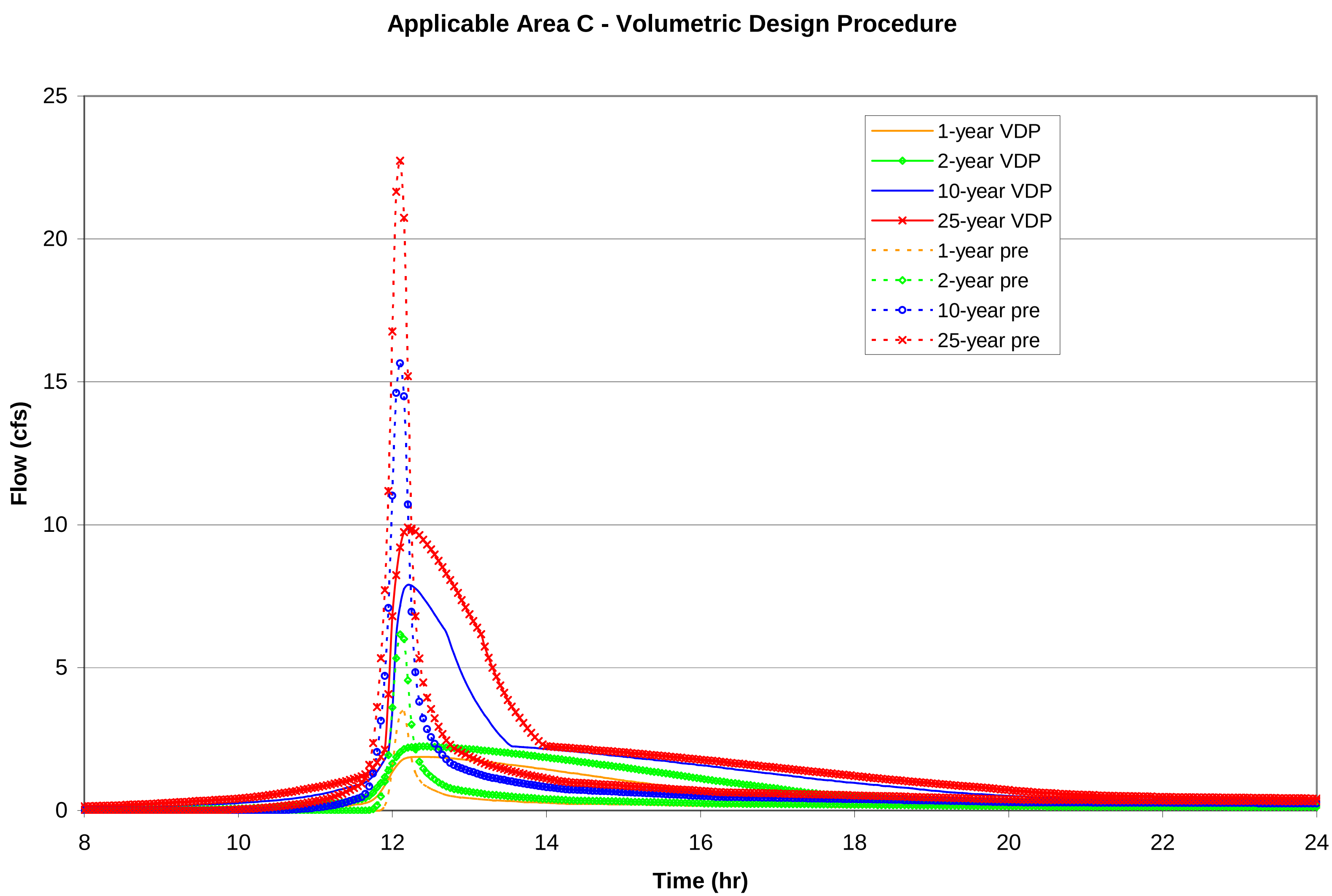
Applicable Area	2-year predevelopment volume released during the critical time period	2-year post-development volume released during the critical time period	25-year predevelopment volume released during the critical time period	25-year post-development volume released during the critical time period
A	11,504	11,223	53,497	52,897
B	3,793	3,476	15,464	14,722
C	5,508	5,231	21,255	19,475
D	3,768	3,400	15,460	14,764
E	9,600	8,539	43,048	41,138
F	2,675	2,484	10,707	10,260
G	1,400	1,278	5,827	5,724
H	1,388	1,181	5,555	5,332
I	5,202	4,945	21,667	21,615



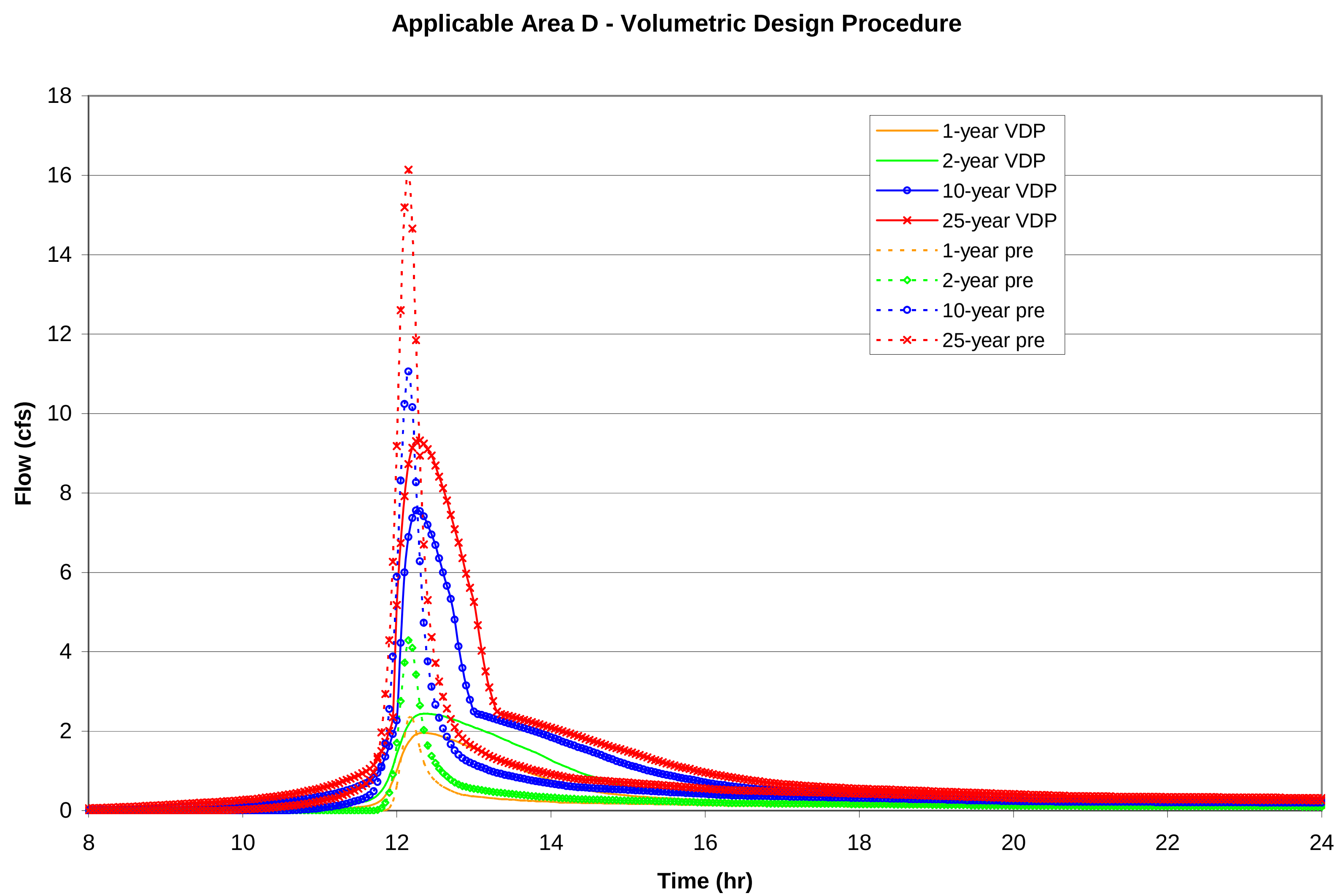
P-3. Post-development hydrographs for applicable area A with the Volumetric Design Procedure design.



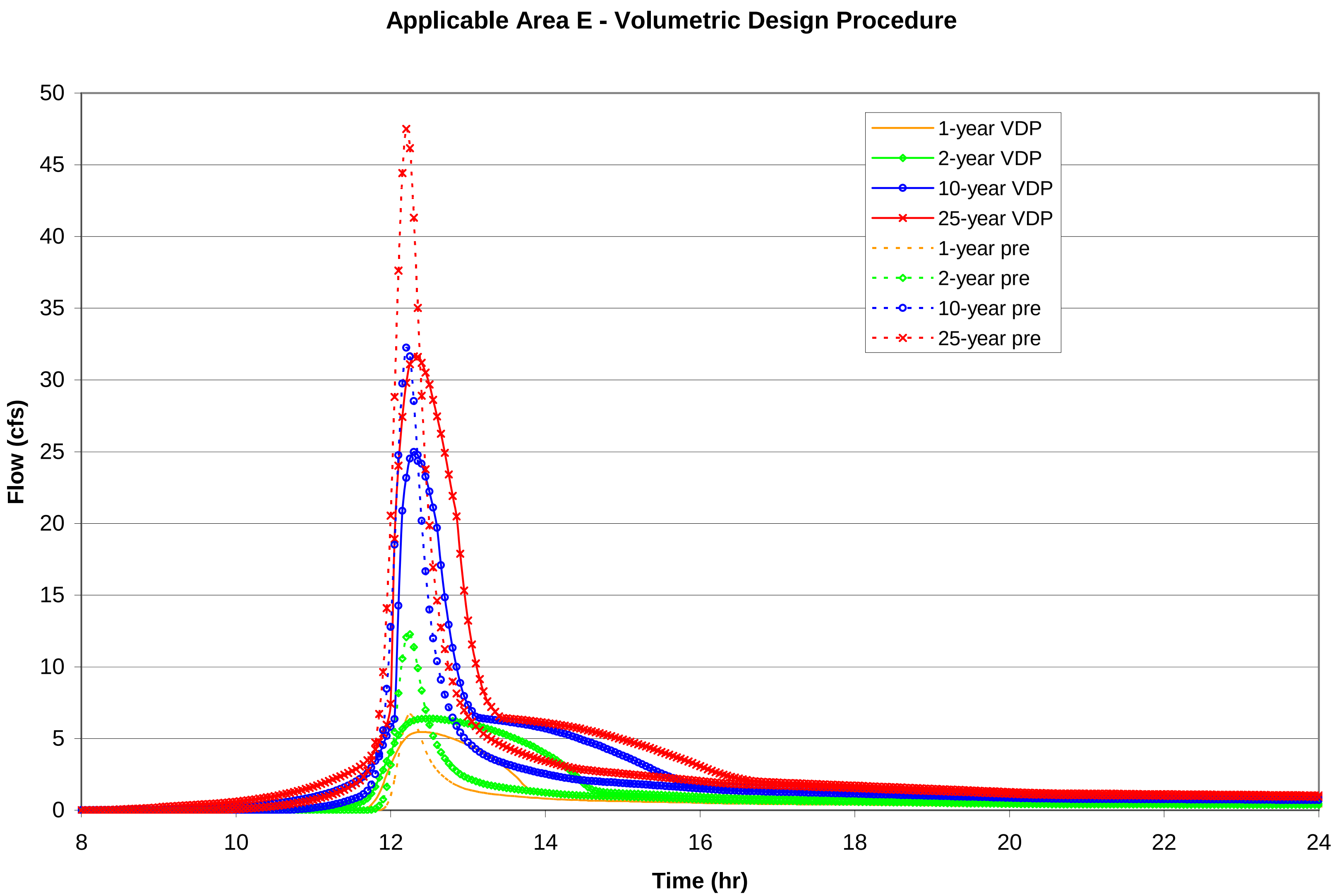
P-4. Post-development hydrographs for applicable area B with the Volumetric Design Procedure design.



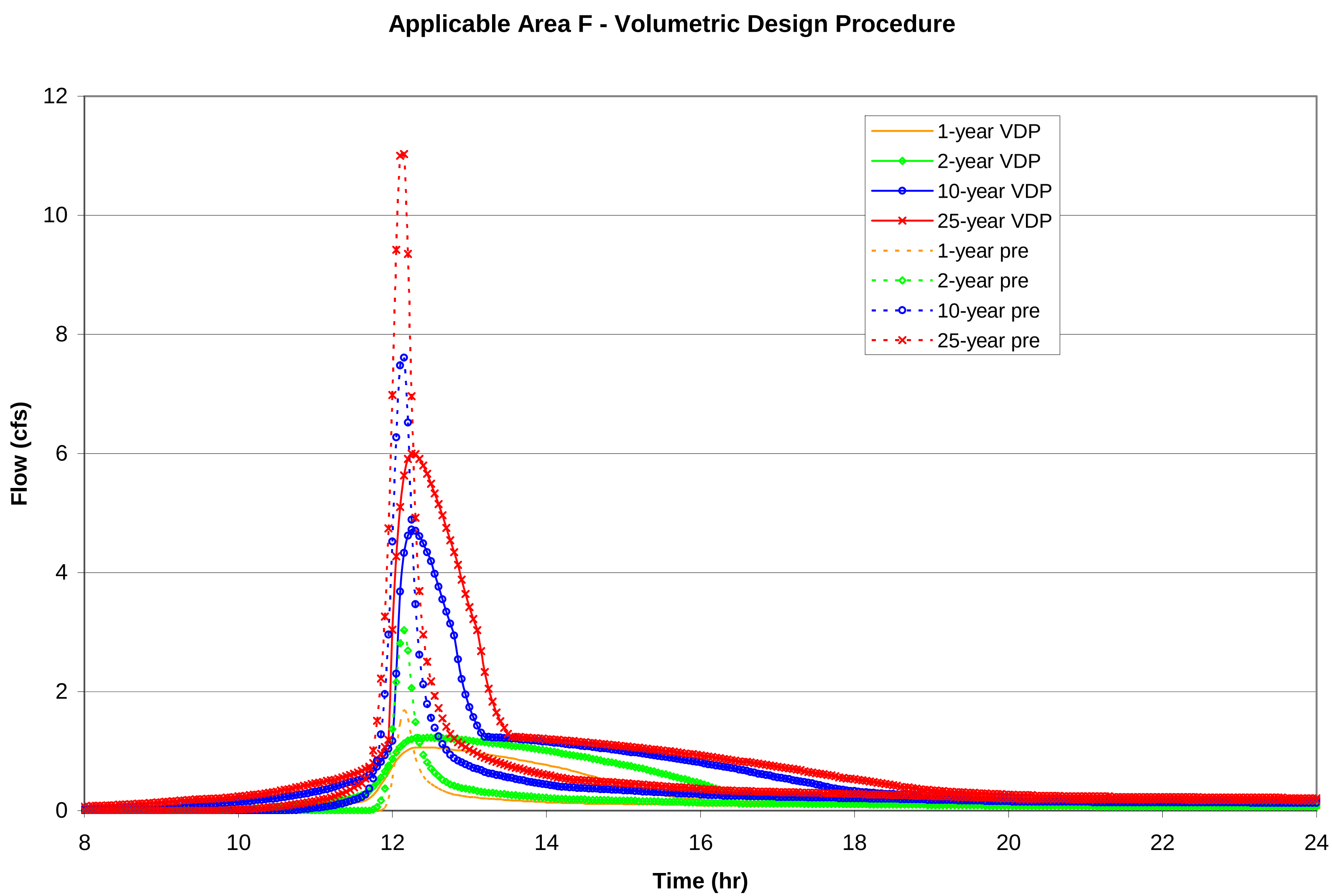
P-5. Post-development hydrographs for applicable area C with the Volumetric Design Procedure design.



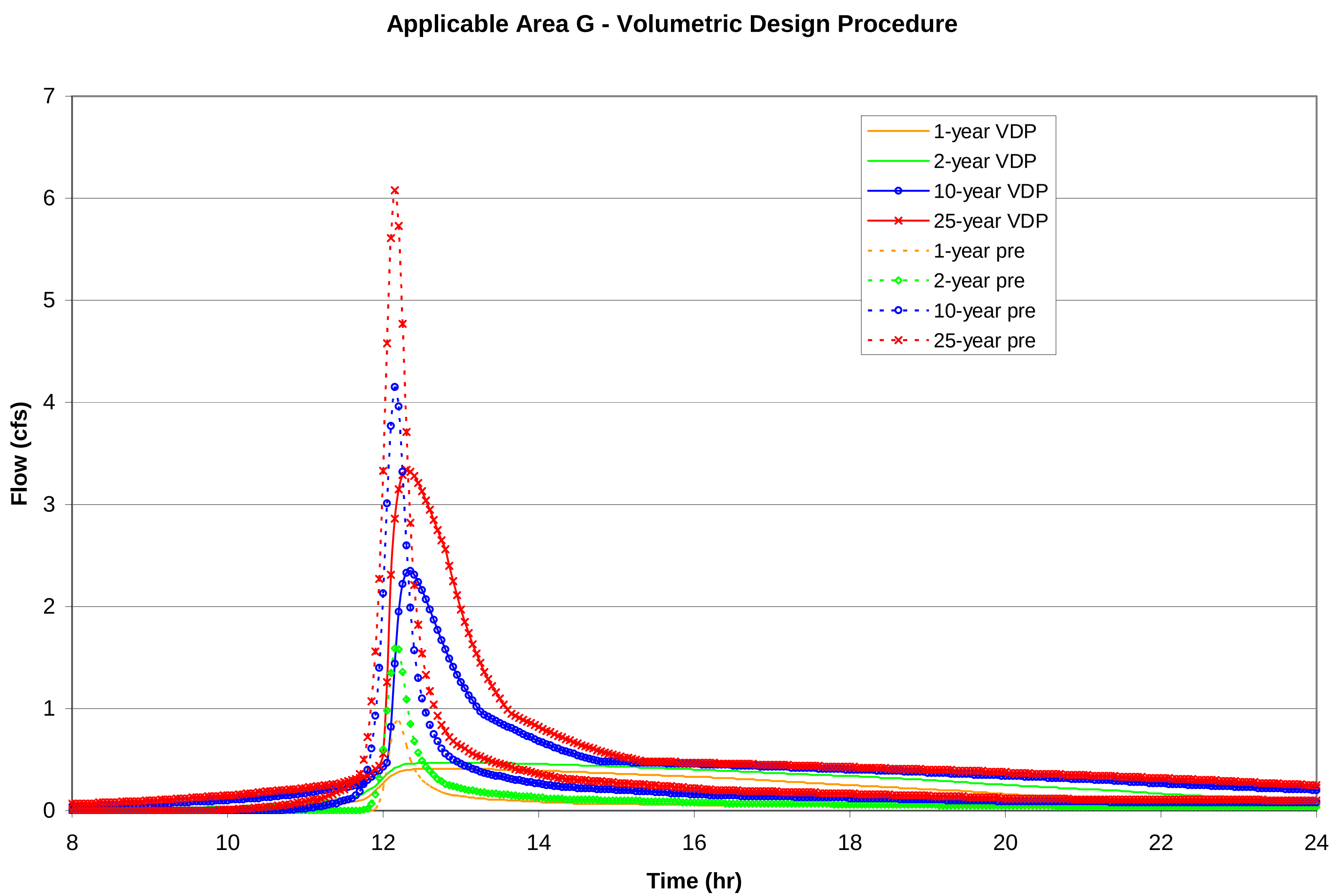
P-6. Post-development hydrographs for applicable area D with the Volumetric Design Procedure design.



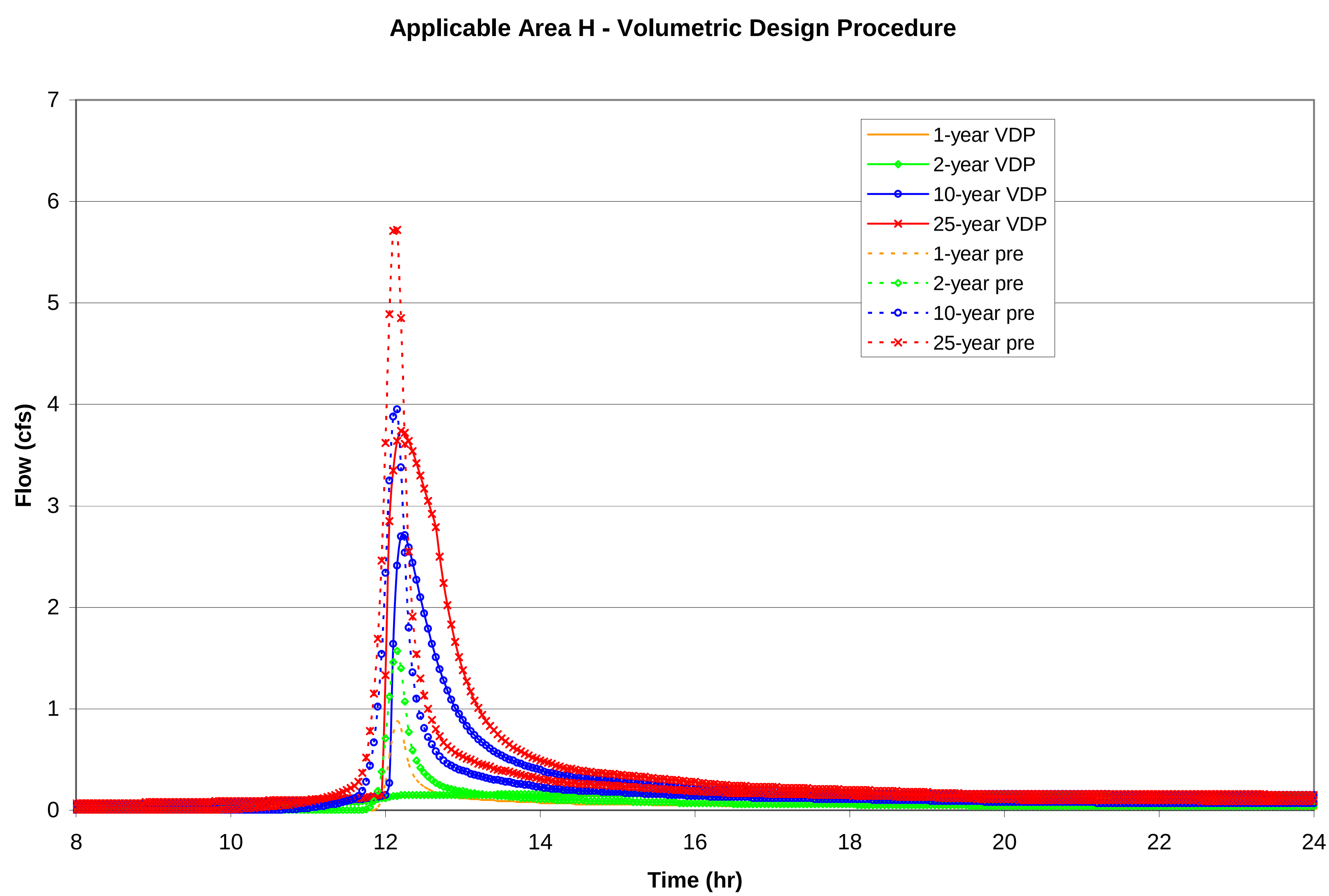
P-7. Post-development hydrographs for applicable area E with the Volumetric Design Procedure design.



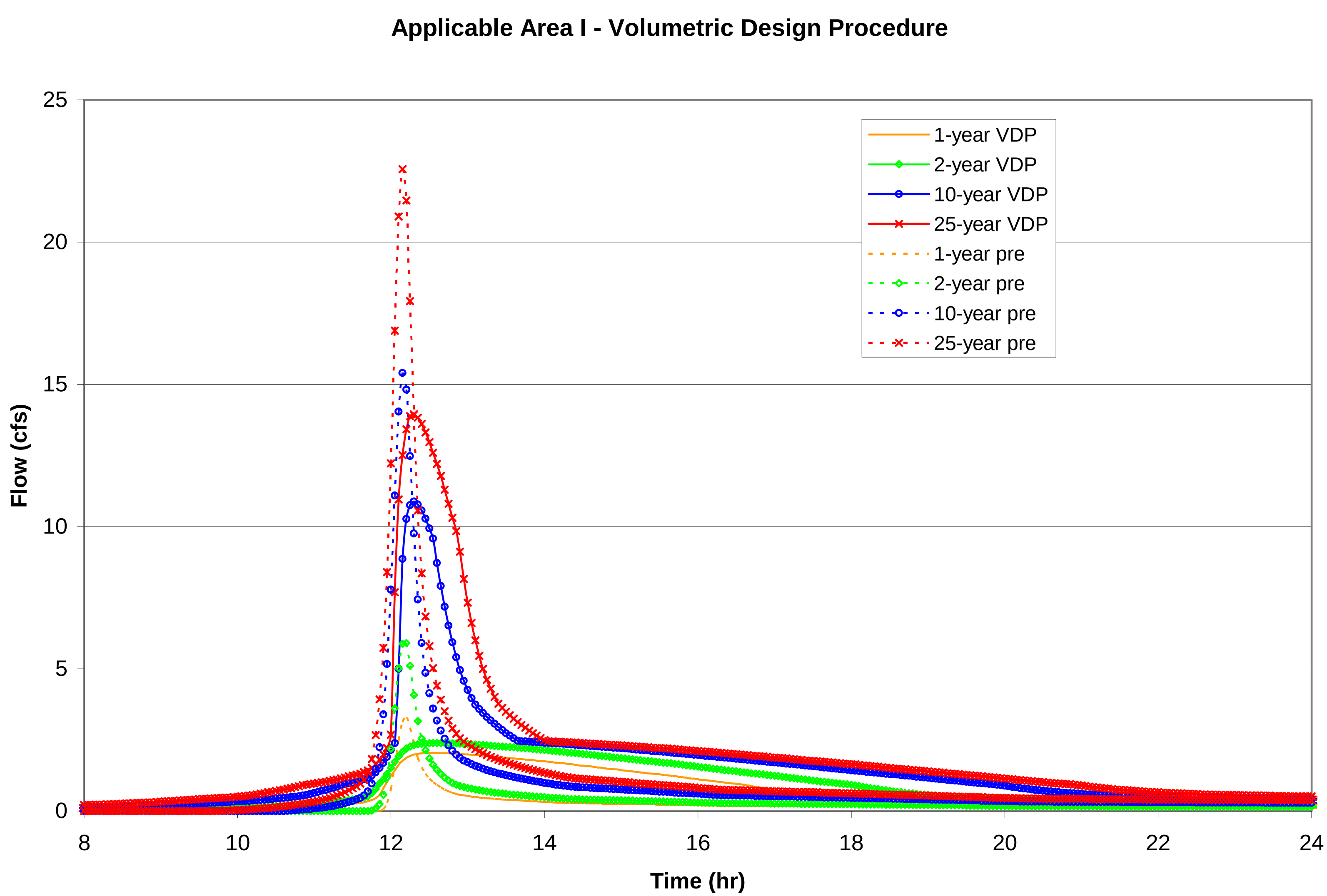
P-8. Post-development hydrographs for applicable area F with the Volumetric Design Procedure design.



P-9. Post-development hydrographs for applicable area G with the Volumetric Design Procedure design.



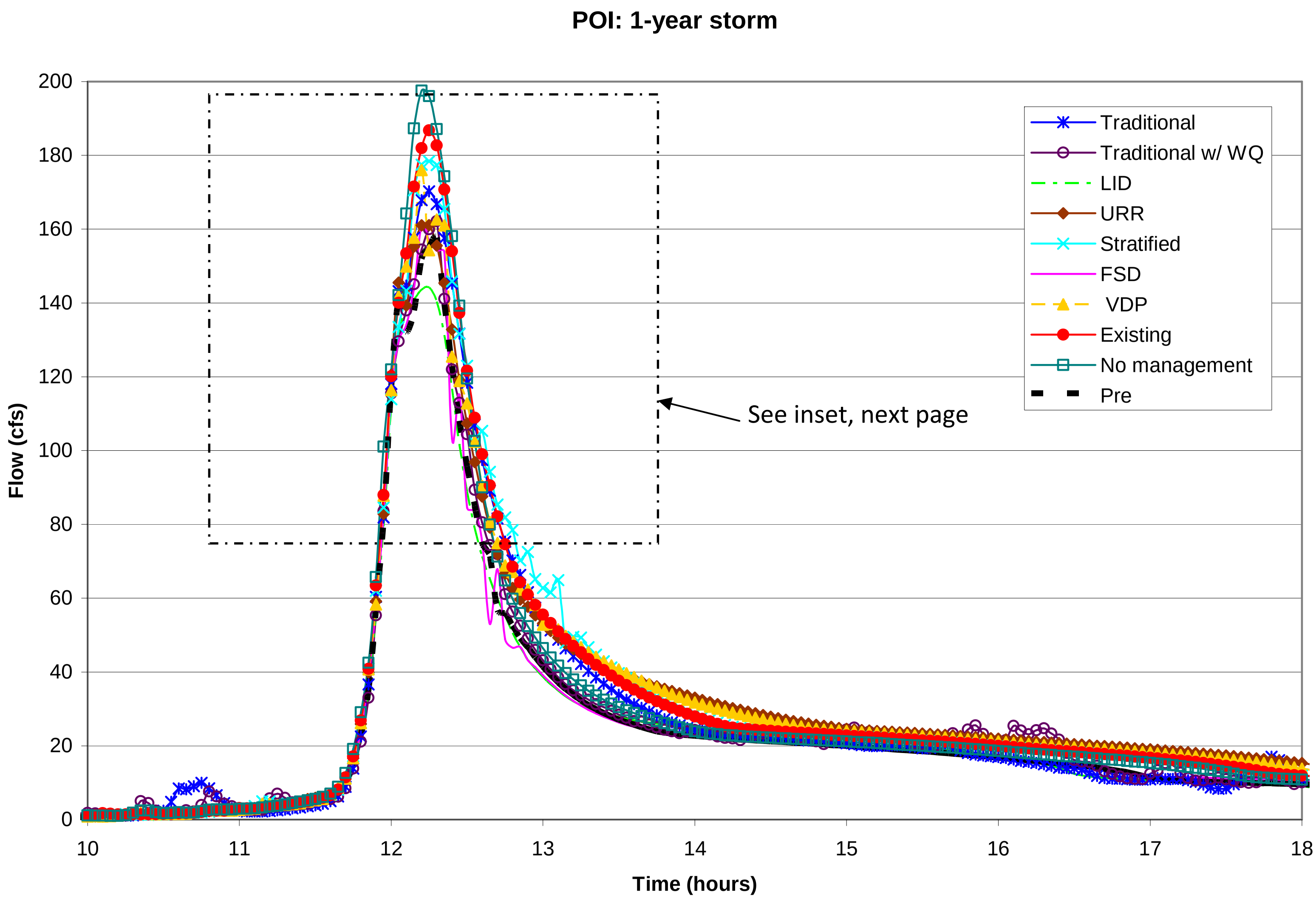
P-10. Post-development hydrographs for applicable area H with the Volumetric Design Procedure design.



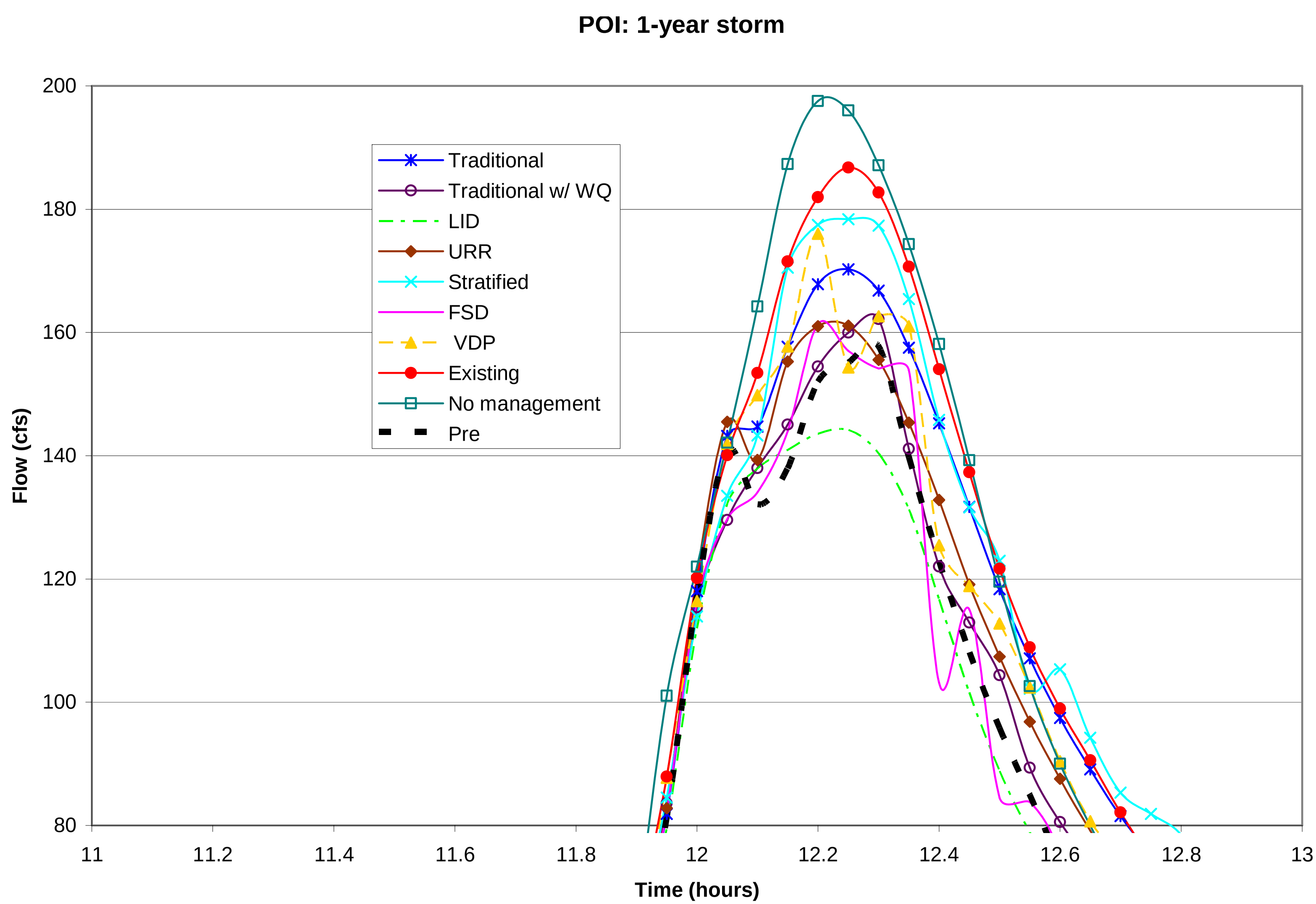
P-11. Post-development hydrographs for applicable area I with the Volumetric Design Procedure design.

Appendix Q: Strategy Comparison Hydrographs at the POI

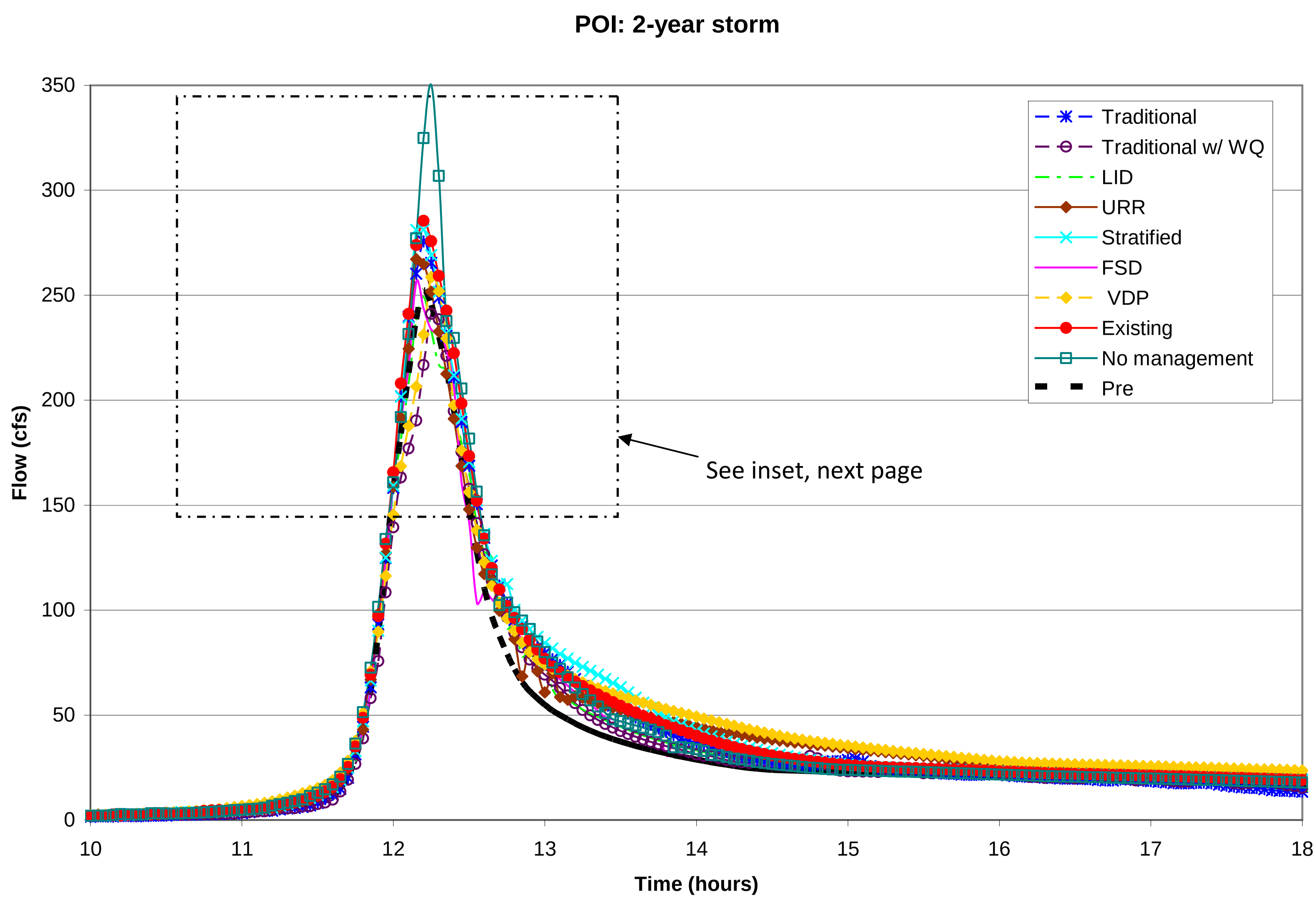
- **Q-1: Comparison of runoff hydrographs for all scenarios at the watershed outfall for the 1-year design storm.**
- **Q-2: Inset from previous page: Comparison of runoff hydrographs for all scenarios at the watershed outfall for the 1-year design storm.**
- **Q-3: Comparison of runoff hydrographs for all scenarios at the watershed outfall for the 2-year design storm.**
- **Q-4: Inset from previous page: Comparison of runoff hydrographs for all scenarios at the watershed outfall for the 2-year design storm.**
- **Q-5: Comparison of runoff hydrographs for all scenarios at the watershed outfall for the 10-year design storm.**
- **Q-6: Inset from previous page: Comparison of runoff hydrographs for all scenarios at the watershed outfall for the 10-year design storm.**
- **Q-7: Comparison of runoff hydrographs for all scenarios at the watershed outfall for the 25-year design storm.**
- **Q-8: Inset from previous page: Comparison of runoff hydrographs for all scenarios at the watershed outfall for the 25-year design storm.**



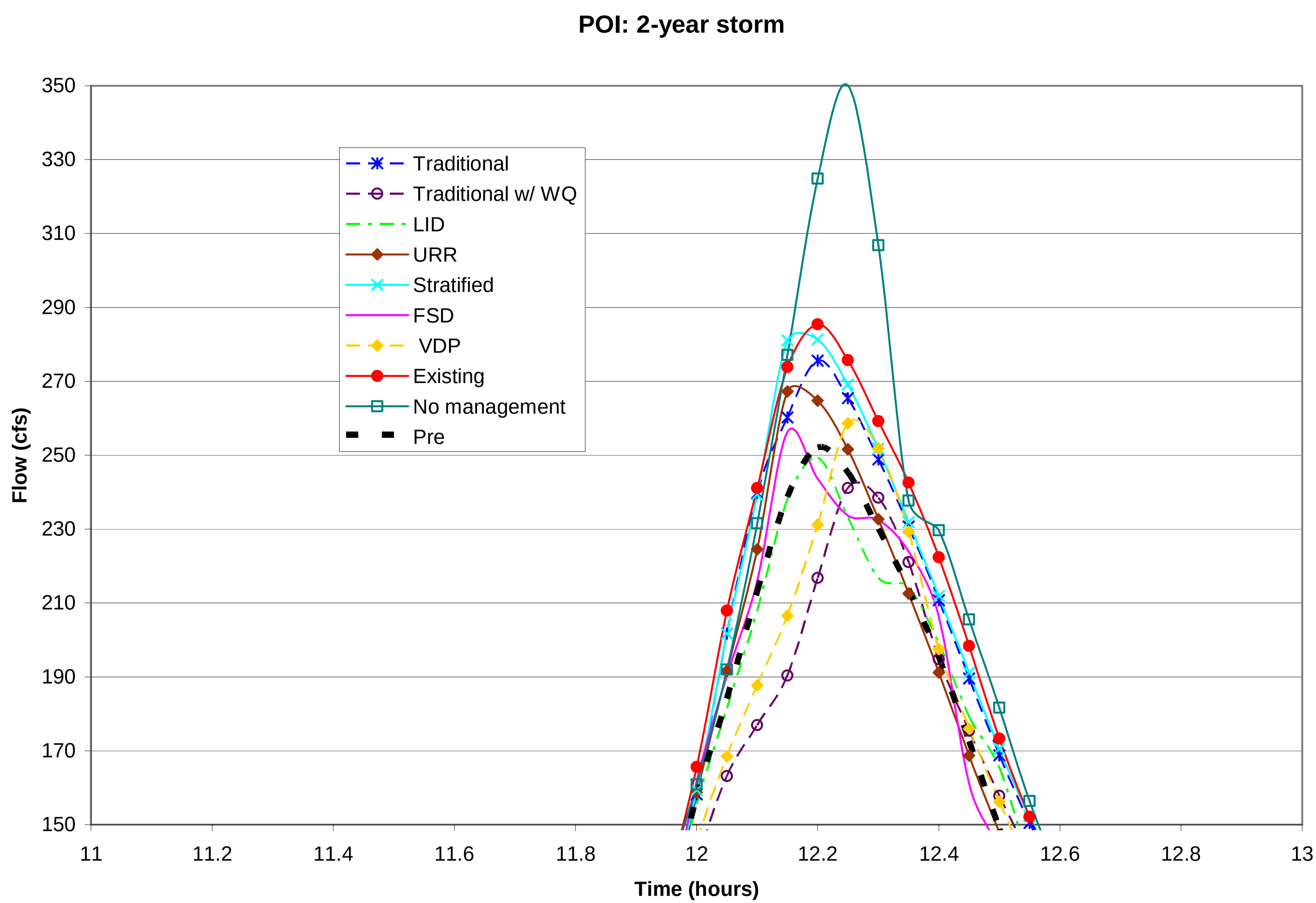
Q-1. Comparison of runoff hydrographs for all scenarios at the watershed outfall for the 1-year design storm.



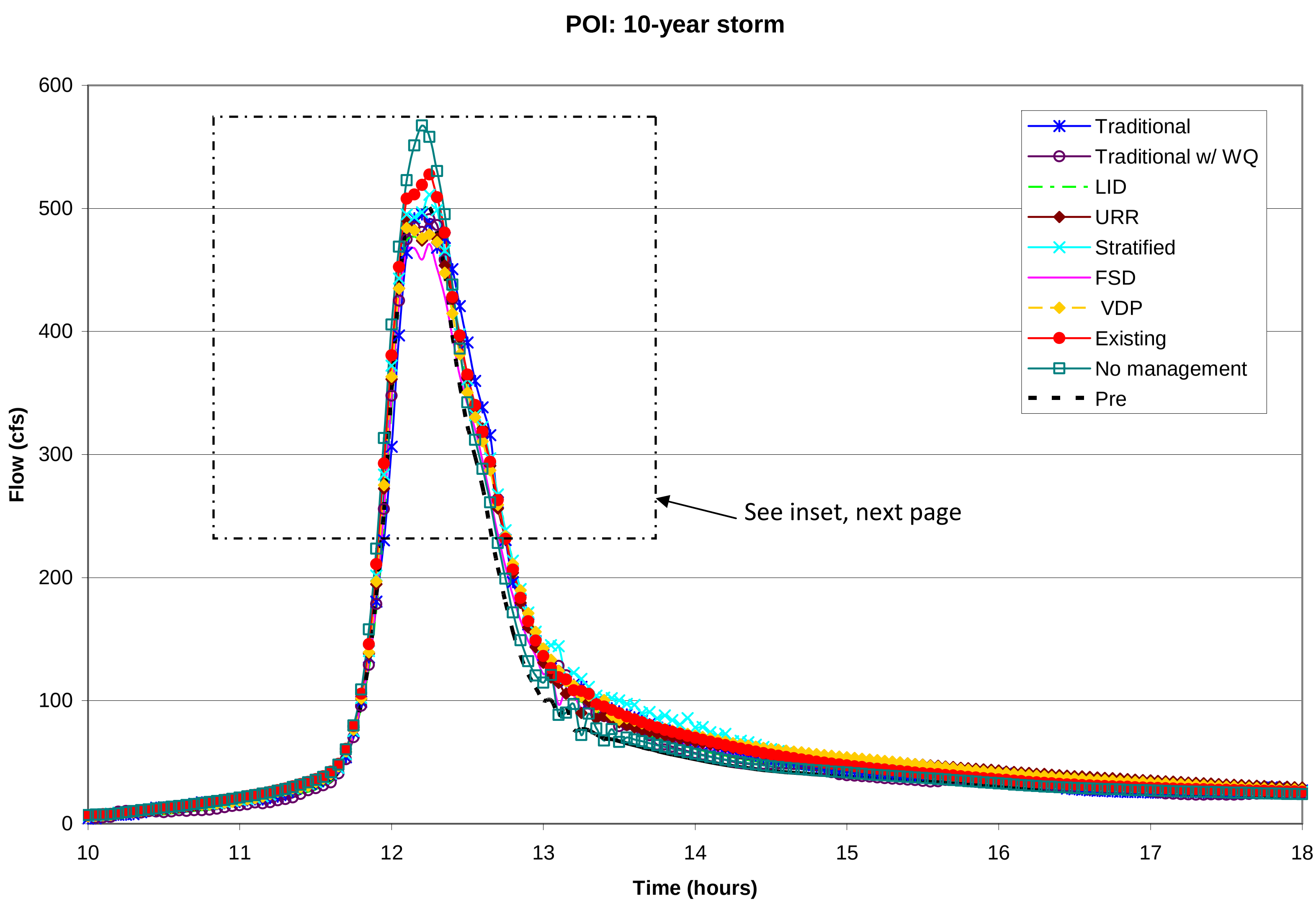
Q-1A. Inset from previous page: Comparison of runoff hydrographs for all scenarios at the watershed outfall for the 1-year design storm.



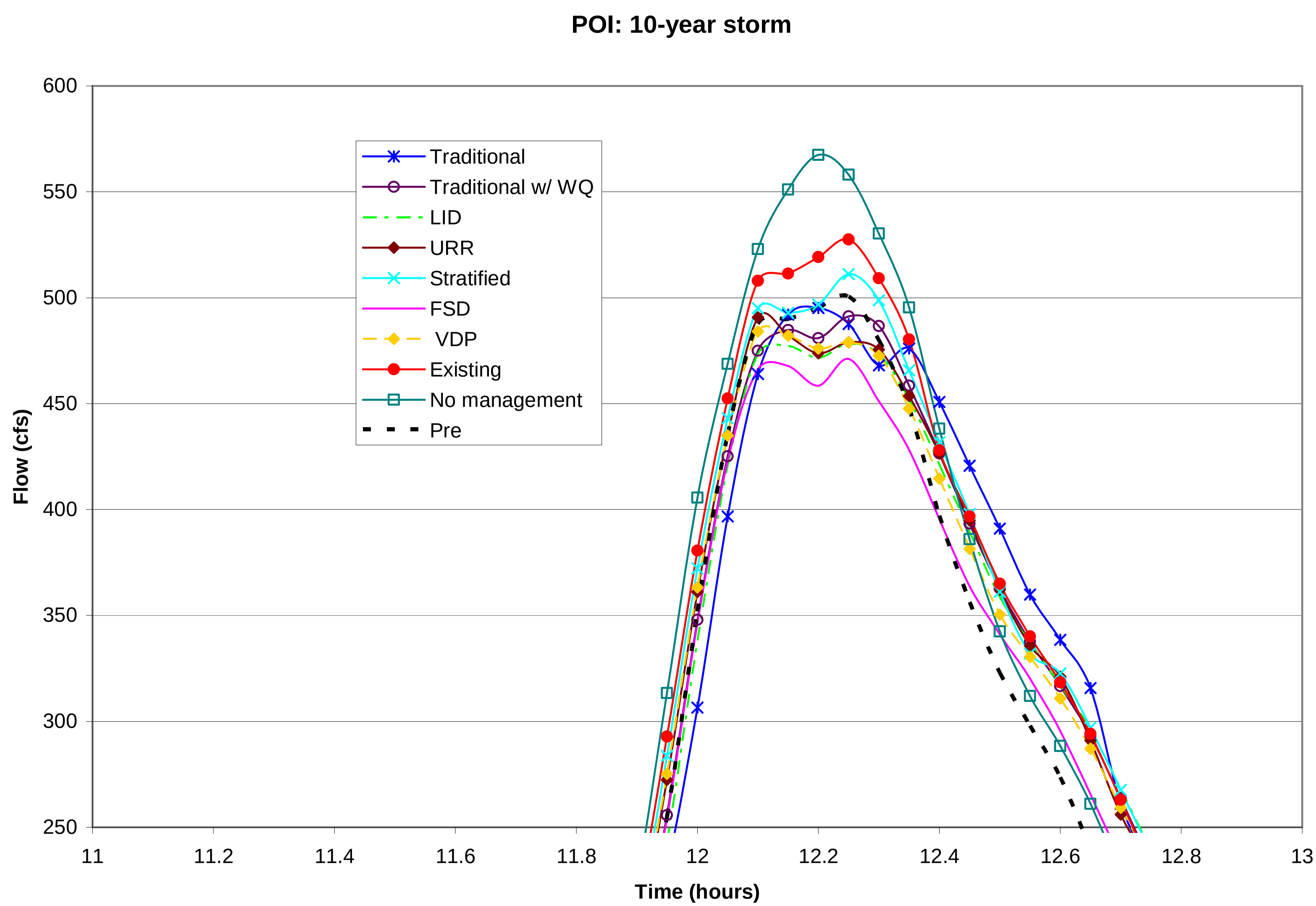
Q-2. Comparison of runoff hydrographs for all scenarios at the watershed outfall for the 2-year design storm.



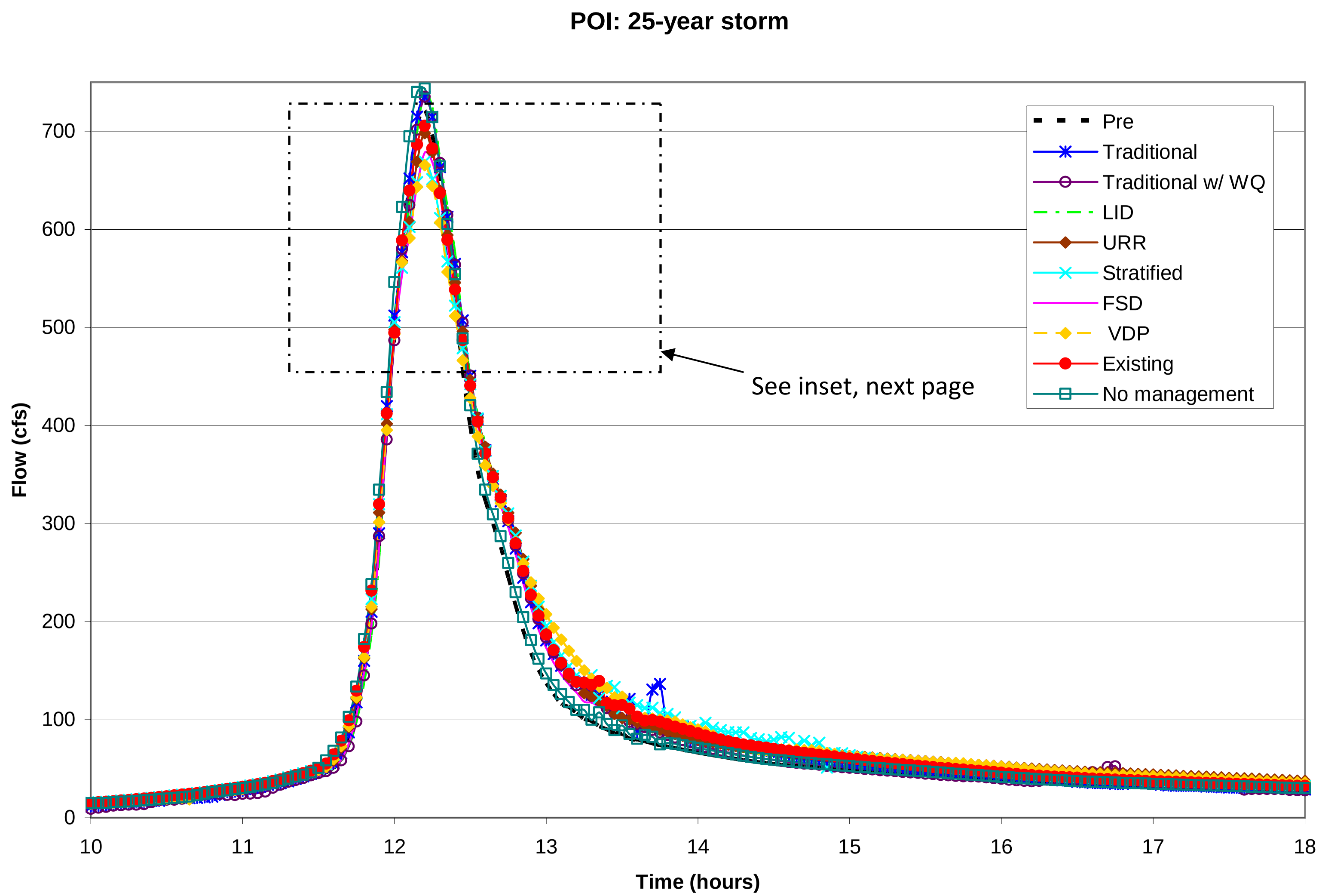
Q-2A. Inset from previous page: Comparison of runoff hydrographs for all scenarios at the watershed outfall for the 2-year design storm.



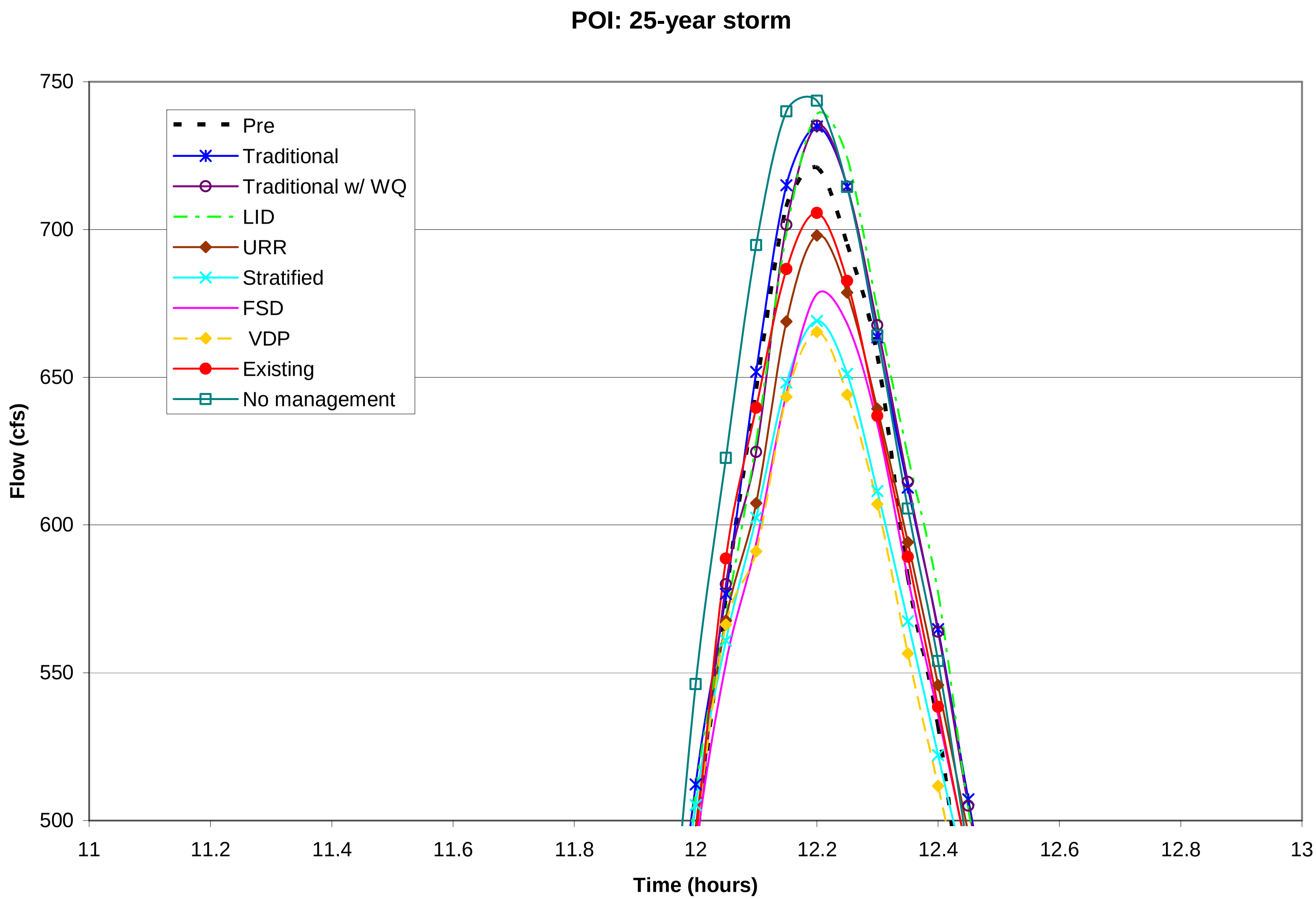
Q-3. Comparison of runoff hydrographs for all scenarios at the watershed outfall for the 10-year design storm.



Q-3A. Inset from previous page: Comparison of runoff hydrographs for all scenarios at the watershed outfall for the 10-year design storm.



Q-4. Comparison of runoff hydrographs for all scenarios at the watershed outfall for the 25-year design storm.



Q-4A. Inset from previous page: Comparison of runoff hydrographs for all scenarios at the watershed outfall for the 25-year design storm.

Appendix R: Information for the Tailored Strategy

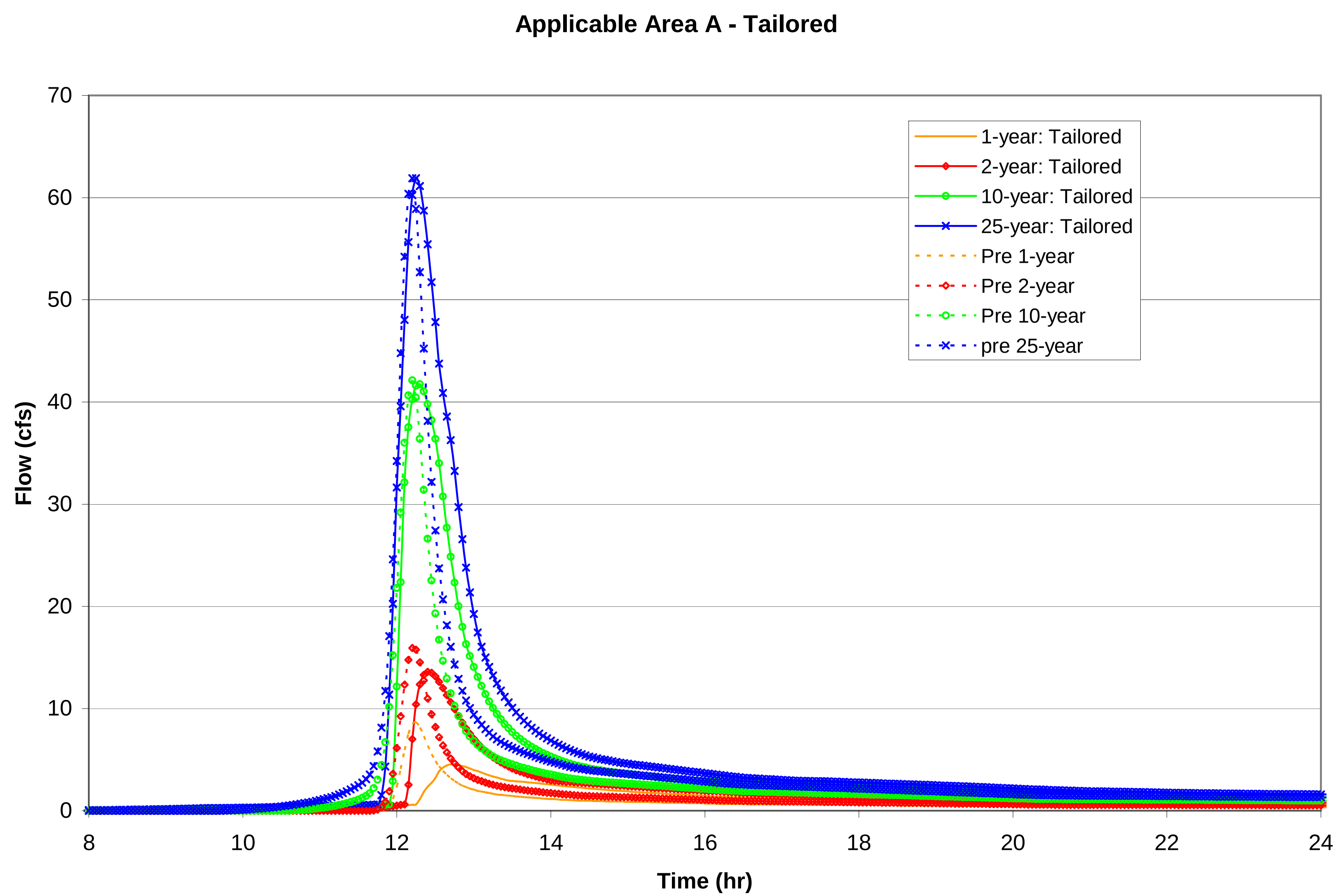
- **R-1: Control Structure information for each facility serving each applicable area used to achieve the Tailored design objectives.**
- **R-2: Comparison of pre- and post-development peak runoff rates for each applicable area with the tailored strategy.**
- **R-3: Post-development hydrographs for applicable area A with the Tailored strategy design.**
- **R-4: Post-development hydrographs for applicable area B with the Tailored strategy design.**
- **R-5: Post-development hydrographs for applicable area C with the Tailored strategy design.**
- **R-6: Post-development hydrographs for applicable area D with the Tailored strategy design.**
- **R-7: Post-development hydrographs for applicable area E with the Tailored strategy design.**
- **R-8: Post-development hydrographs for applicable area F with the Tailored strategy design.**
- **R-9: Post-development hydrographs for applicable area G with the Tailored strategy design.**
- **R-10: Post-development hydrographs for applicable area H with the Tailored strategy design.**
- **R-11: Post-development hydrographs for applicable area I with the Tailored strategy design.**

R-1. Control Structure information for each facility serving each applicable area used to achieve the Tailored design objectives.

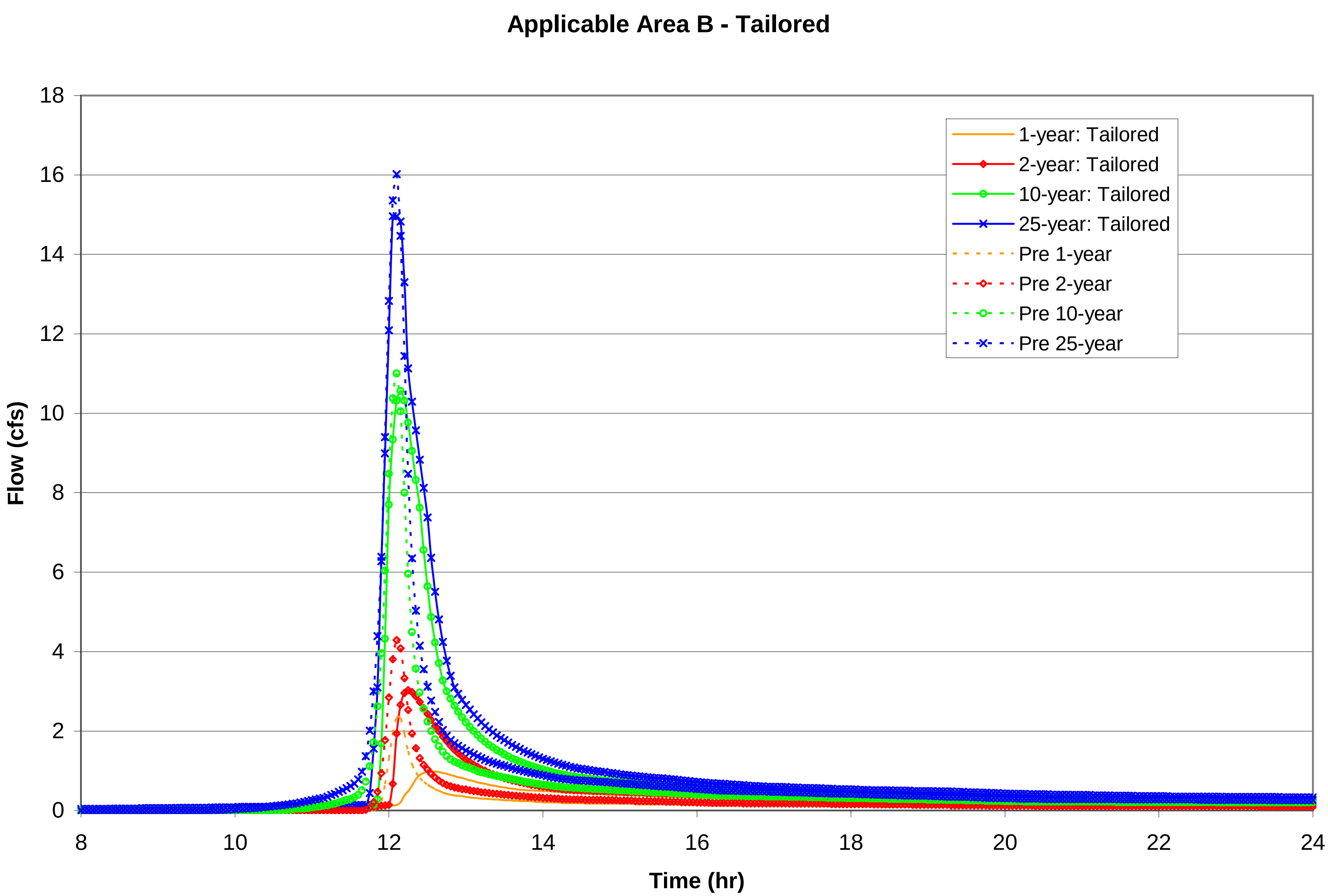
Applicable Area	WQ Orifice size		Primary Orifice		Secondary Orifice		Riser	
	Dia (in)	Invert (ft)	Size	Invert (ft)	Size	Crest (ft)	Size	Crest (ft)
A	3.5	2,165.00	24" Dia.	2,169.00	N/A		24"	2,171.90
B	1.8	2,238.10	18" Dia.	2,241.40	N/A		24"	2,243.70
C	2.1	2,204.00	20'W x 6'H	2,206.75	N/A		30"	2,207.85
D	1.3	2,160.00	13" Dia	2,161.82	N/A		18"	2,163.20
E	0.5	2,137.26	15" Dia	2,138.80	23"	2,143.00	N/A	
F	1.2	2,121.30	12" Dia	2,123.90	N/A		7.5"	2,125.10
G	1.5	2,110.50	12" Dia	2,112.26	N/A		3.5"	2,113.00
H	1.1	2,094.20	7" Dia	2,099.05	N/A		8"	2,100.40
I	2.5	2,078.50	21" Dia	2,080.16	N/A		18"	2,082.40

R-2. Comparison of pre- and post-development peak runoff rates for each applicable area with the tailored strategy.

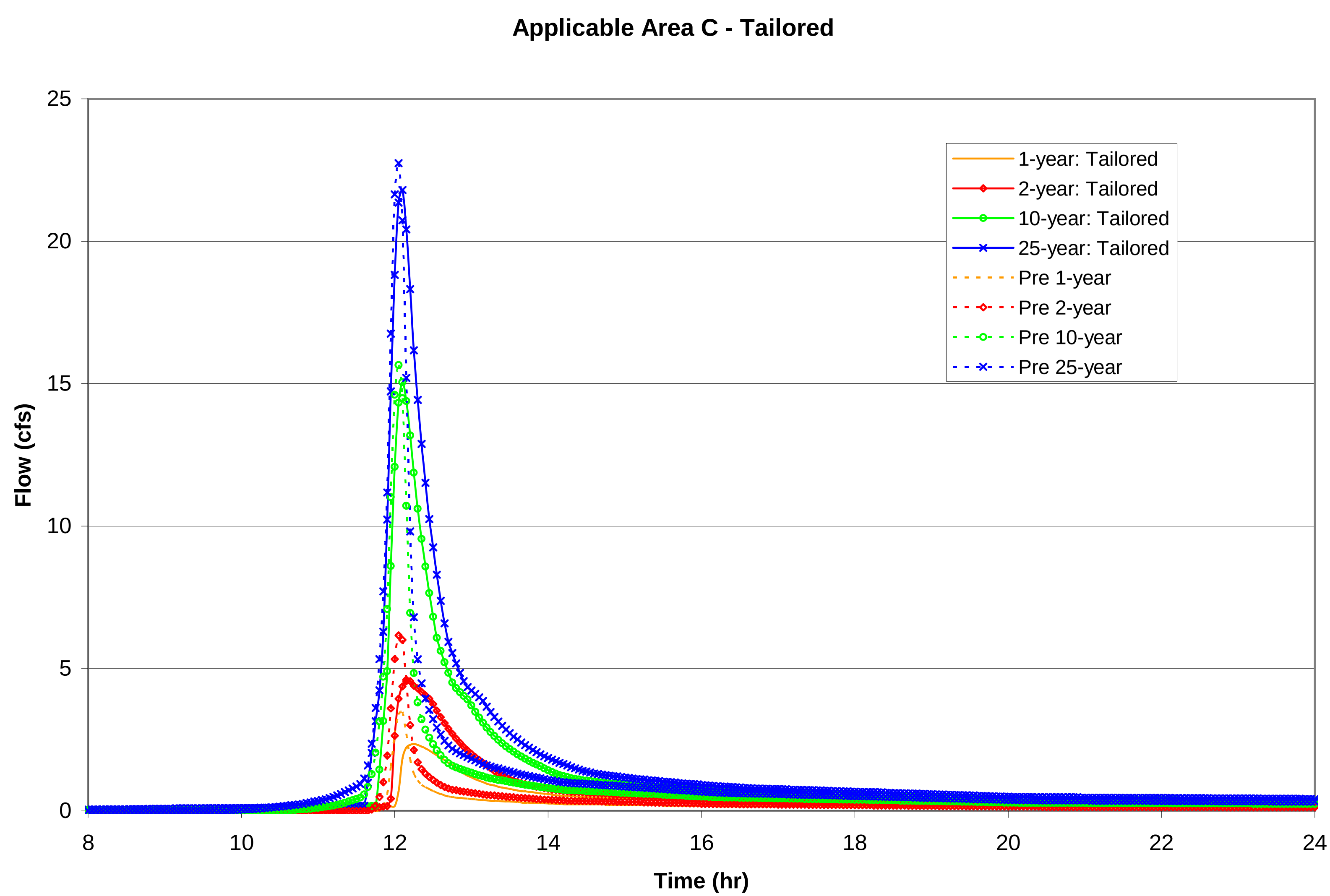
Applicable Area	1-year		2-year		10-year		25-year	
	pre peak (cfs)	post-peak (cfs)	pre peak (cfs)	post-peak (cfs)	pre peak (cfs)	post-peak (cfs)	pre peak (cfs)	post-peak (cfs)
A	4.64	4.54	15.92	13.59	42.13	41.75	61.9	61.90
B	2.32	0.99	4.29	3.03	11.00	10.56	16.02	14.96
C	3.47	2.35	6.16	4.61	15.65	15.05	22.74	21.80
D	2.33	1.94	4.29	3.98	11.06	10.85	16.14	12.47
E	6.70	8.58	12.28	11.08	32.25	31.96	47.50	37.95
F	1.69	1.50	3.03	3.03	7.61	7.54	11.03	10.54
G	0.88	0.58	1.59	1.45	4.15	4.15	6.08	5.24
H	0.88	0.71	1.57	1.32	3.95	3.86	5.72	4.30
I	3.30	2.18	5.91	4.88	15.4	13.49	22.57	21.96



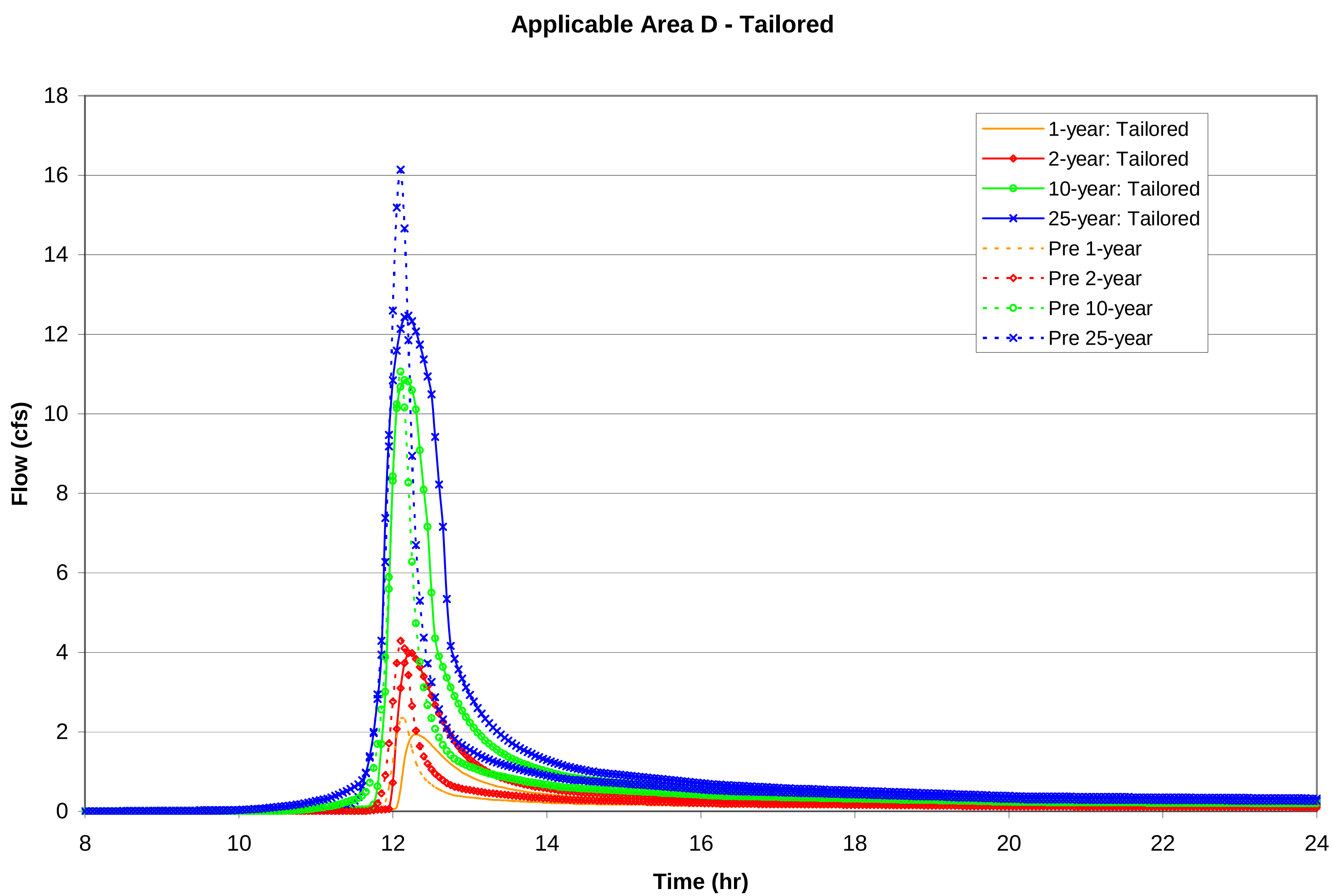
R-3. Post-development hydrographs for applicable area A with the Tailored strategy design.



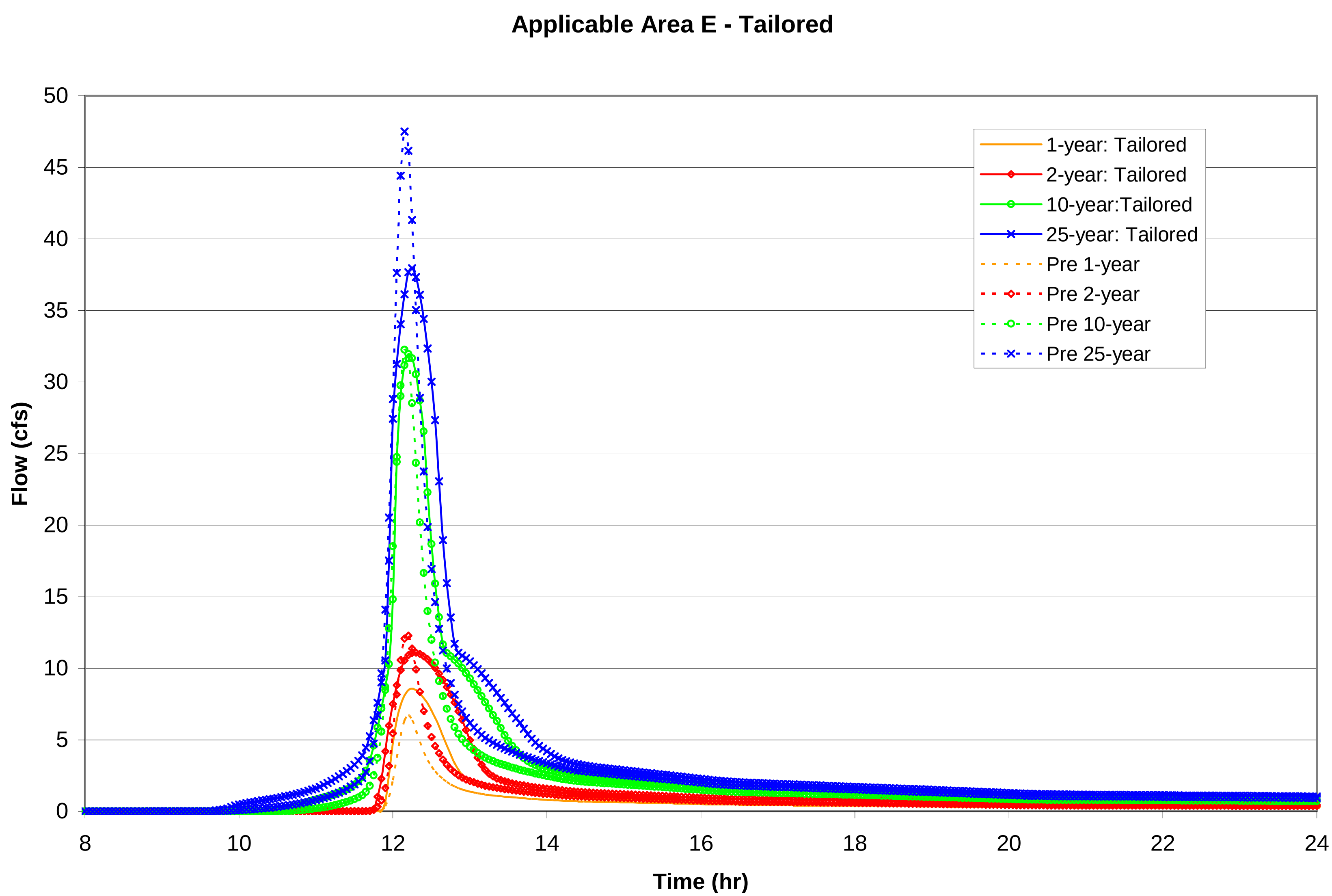
R-4. Post-development hydrographs for applicable area B with the Tailored strategy design.



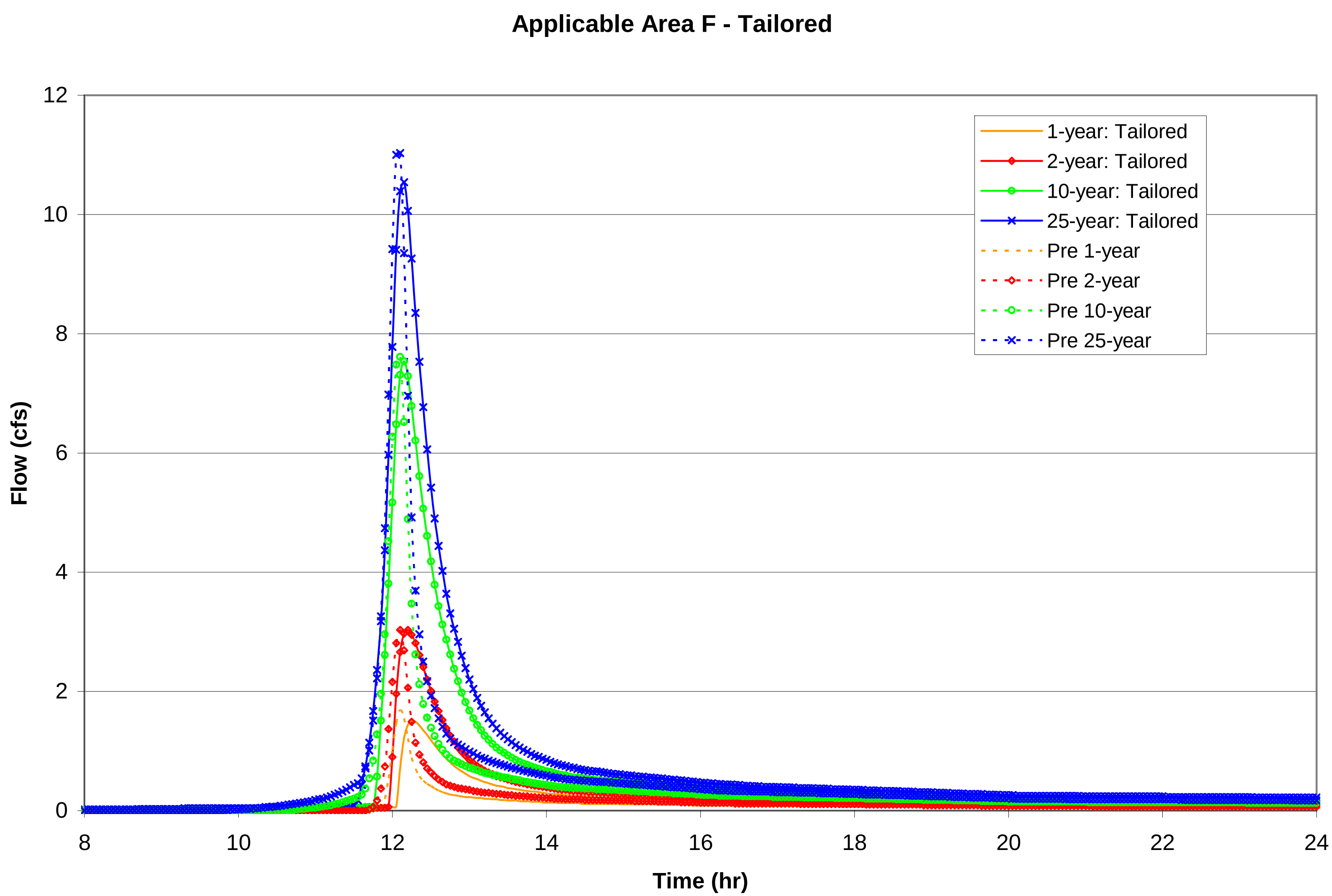
R-5. Post-development hydrographs for applicable area C with the Tailored strategy design.



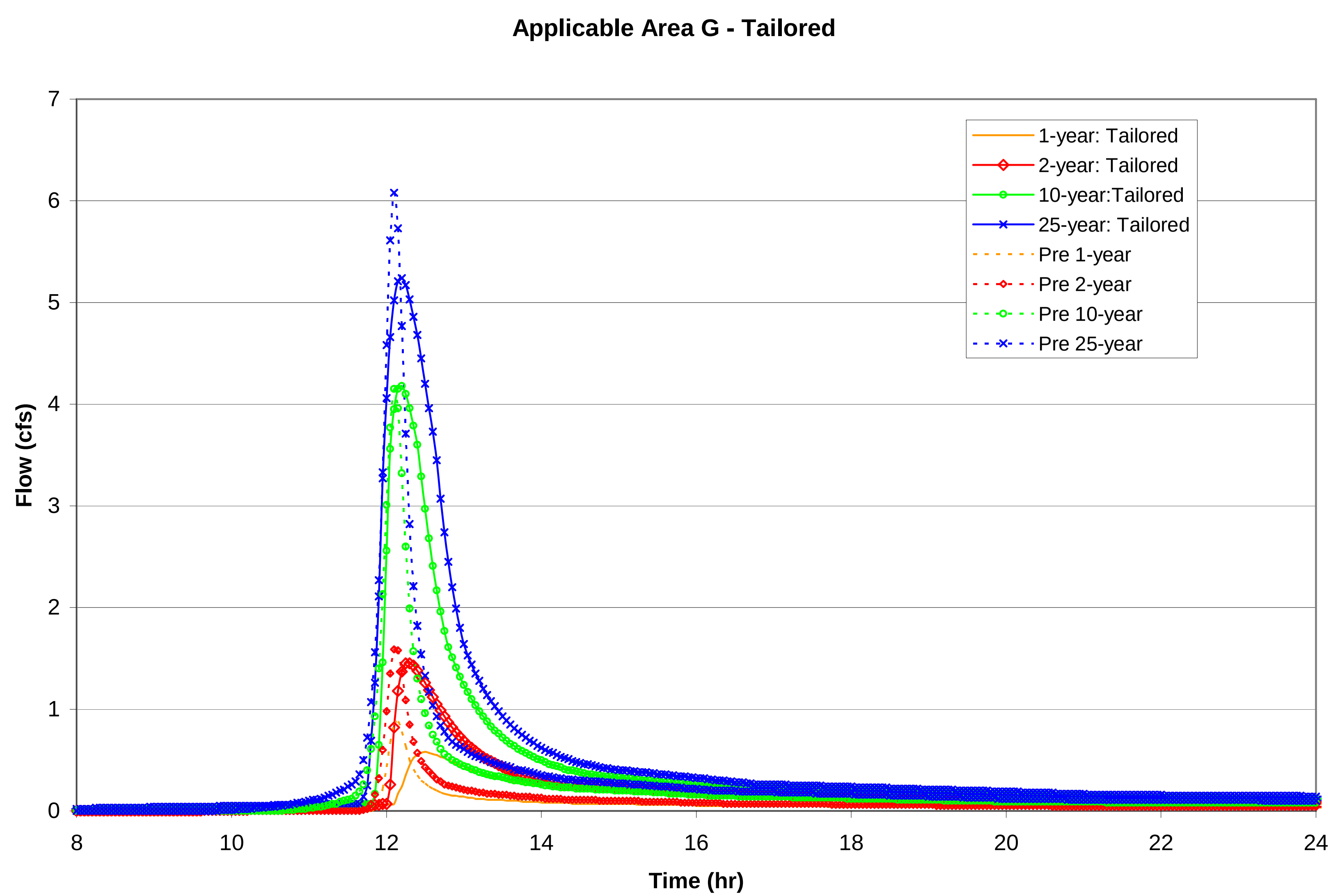
R-6. Post-development hydrographs for applicable area D with the Tailored strategy design.



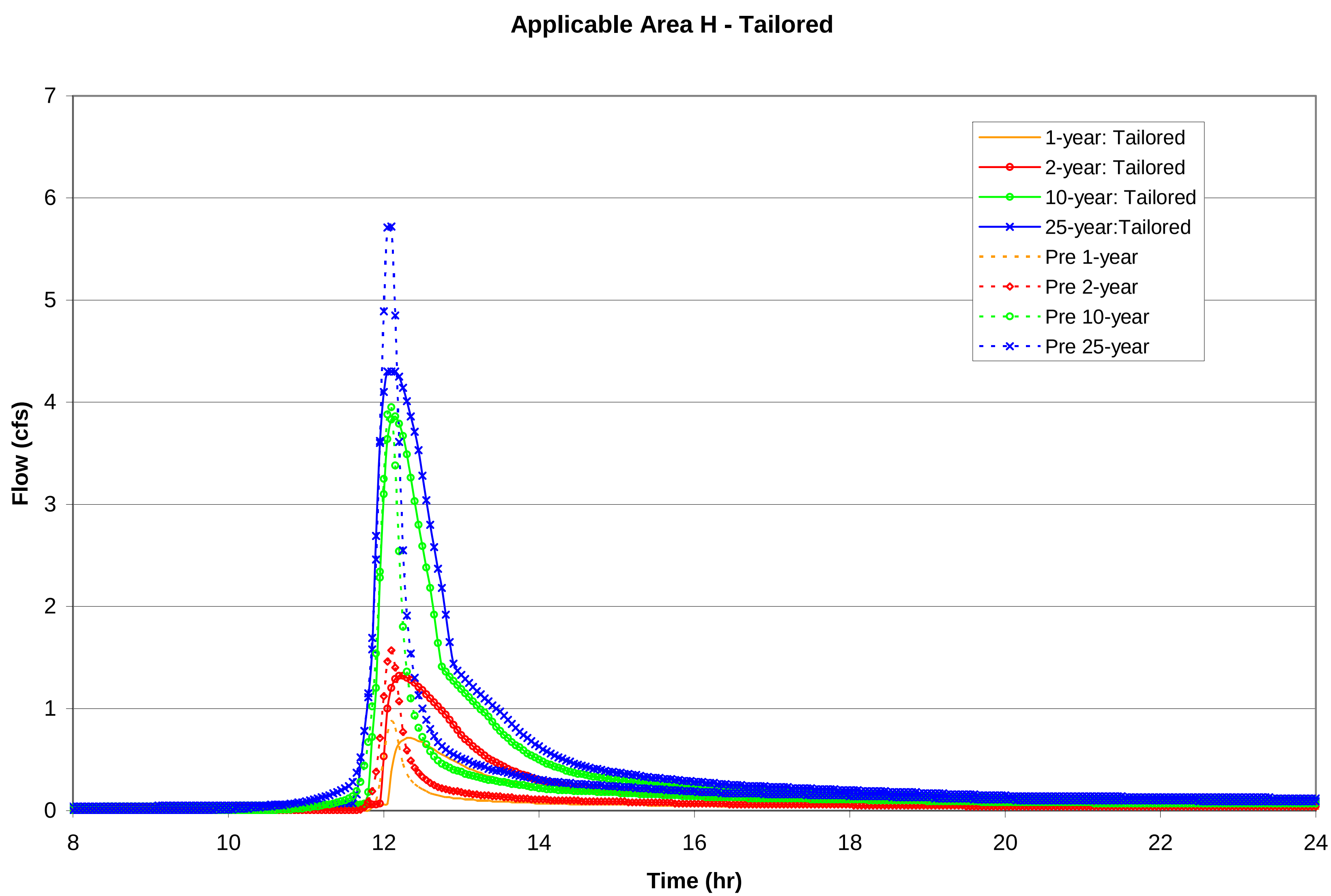
R-7. Post-development hydrographs for applicable area E with the Tailored strategy design.



R-8. Post-development hydrographs for applicable area F with the Tailored strategy design.

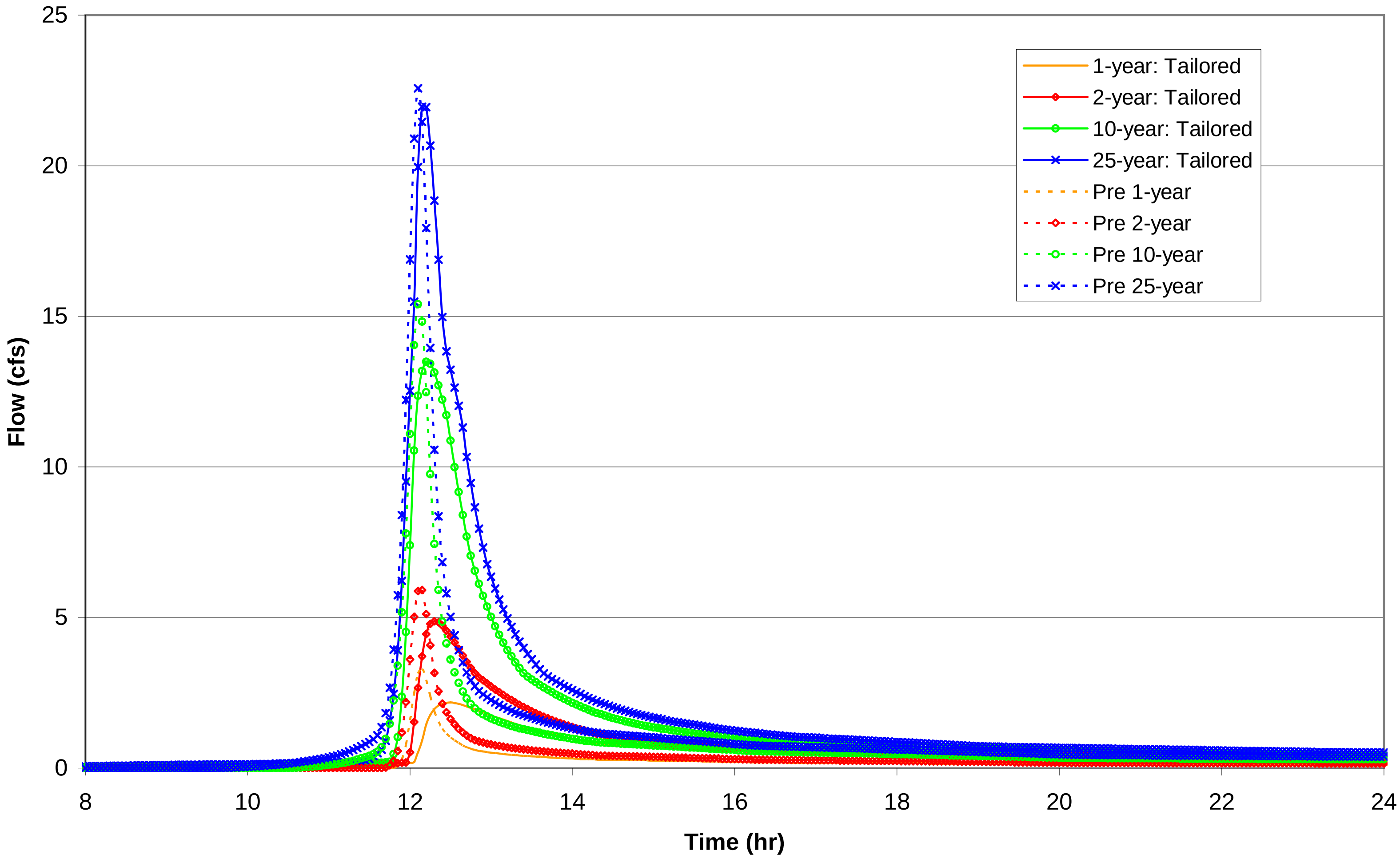


R-9. Post-development hydrographs for applicable area G with the Tailored strategy design.



R-10. Post-development hydrographs for applicable area H with the Tailored strategy design.

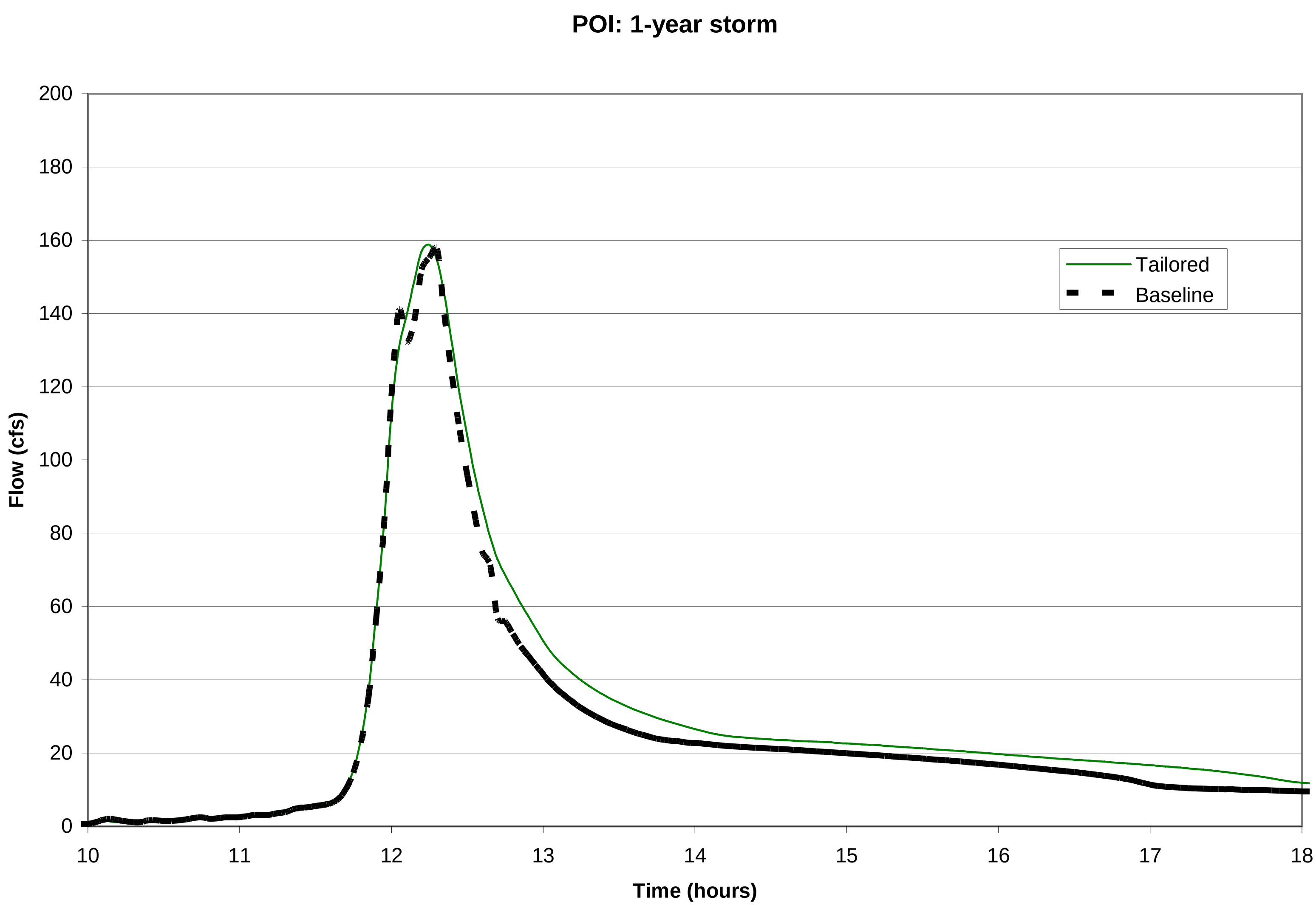
Applicable Area I - Tailored



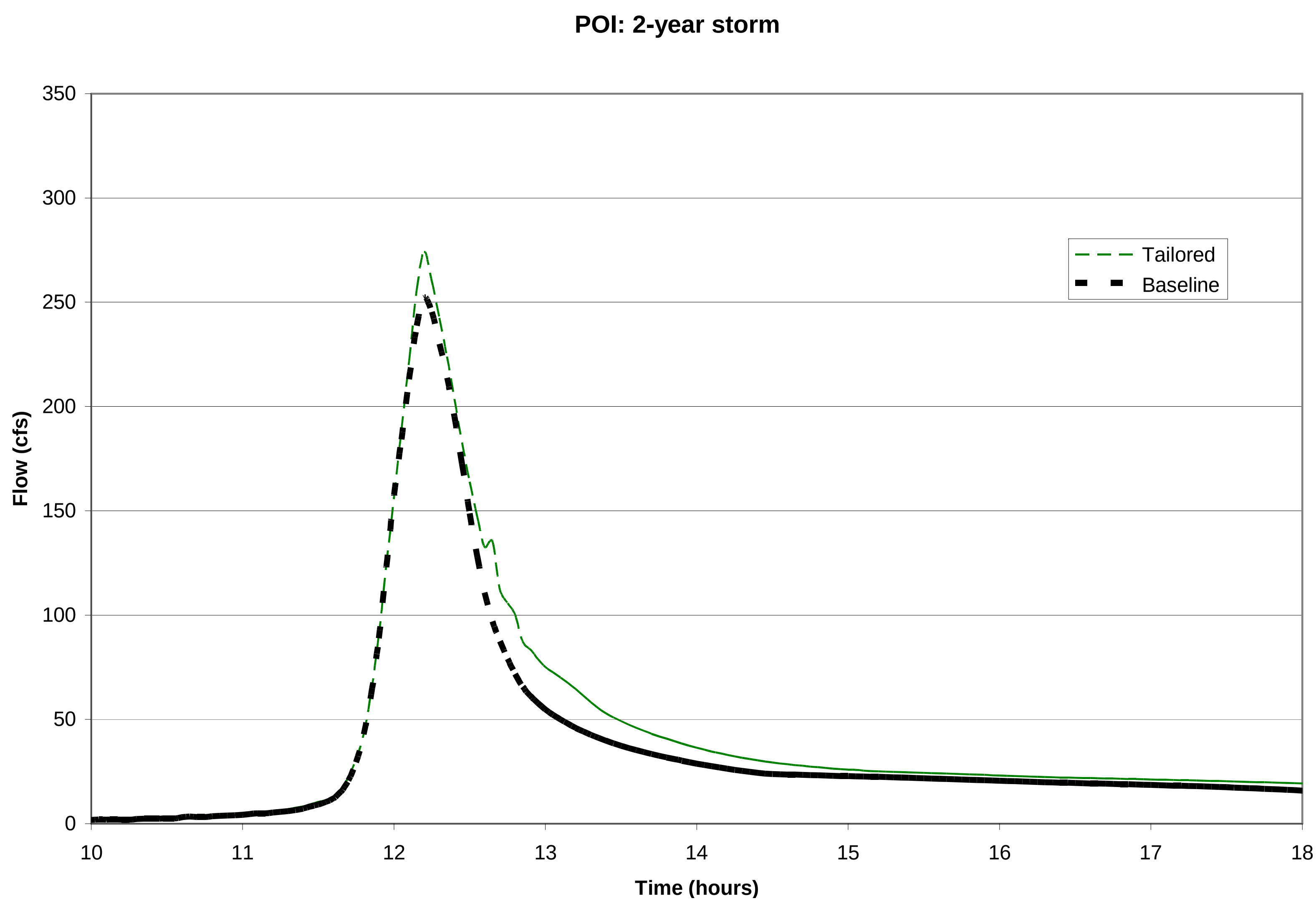
R-11. Post-development hydrographs for applicable area I with the Tailored strategy design.

Appendix S: Watershed Runoff hydrographs for the Tailored Strategy

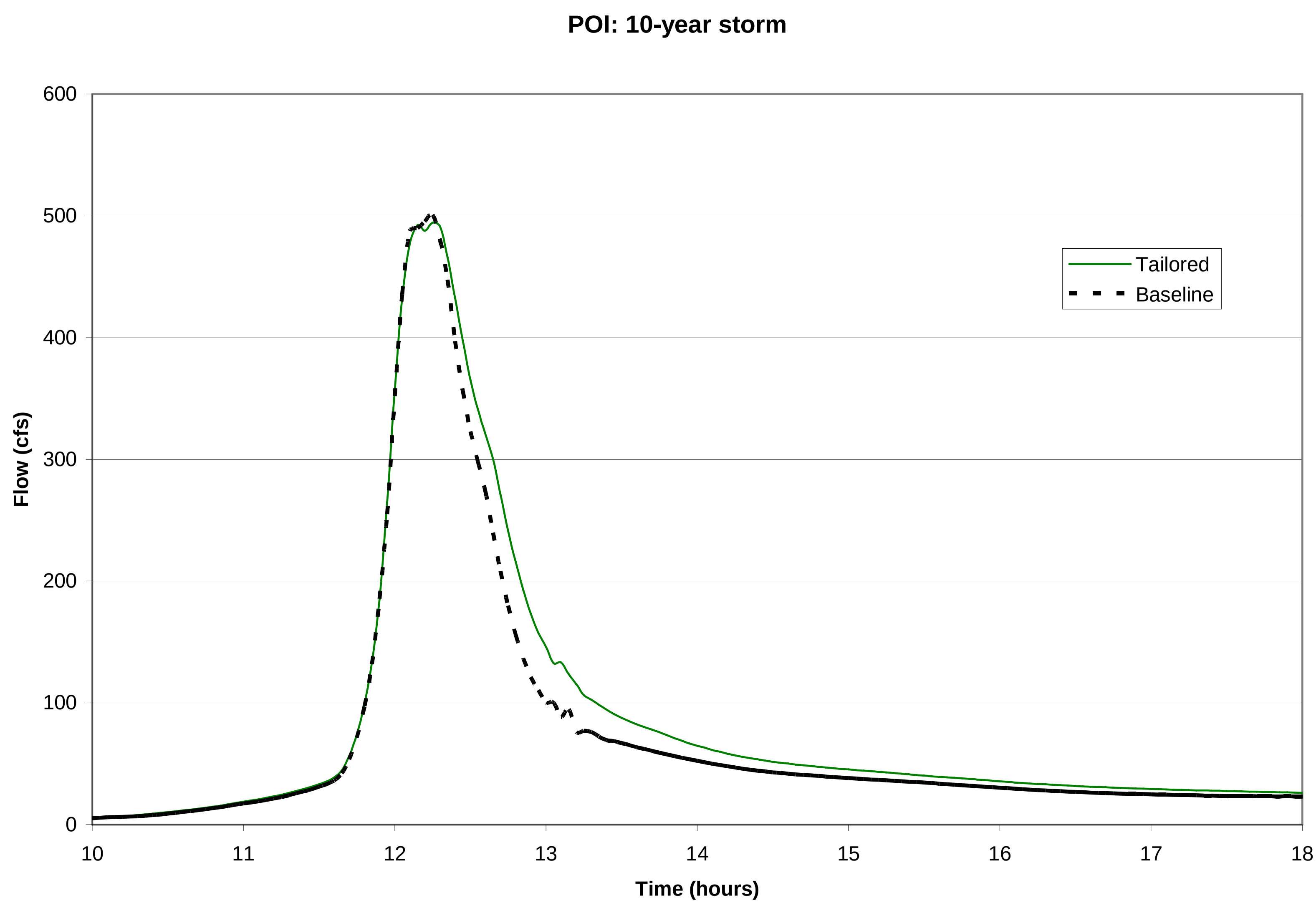
- **S-1: Comparison of runoff hydrographs for the overall watershed resulting from implementation of the Tailored strategy for the 1-year design storm, compared to the watershed baseline hydrographs.**
- **S-2: Comparison of runoff hydrographs for the overall watershed resulting from implementation of the Tailored strategy for the 2-year design storm, compared to the watershed baseline hydrographs.**
- **S-3: Comparison of runoff hydrographs for the overall watershed resulting from implementation of the Tailored strategy for the 10-year design storm, compared to the watershed baseline hydrographs.**
- **S-4: Comparison of runoff hydrographs for the overall watershed resulting from implementation of the Tailored strategy for the 25-year design storm, compared to the watershed baseline hydrographs.**



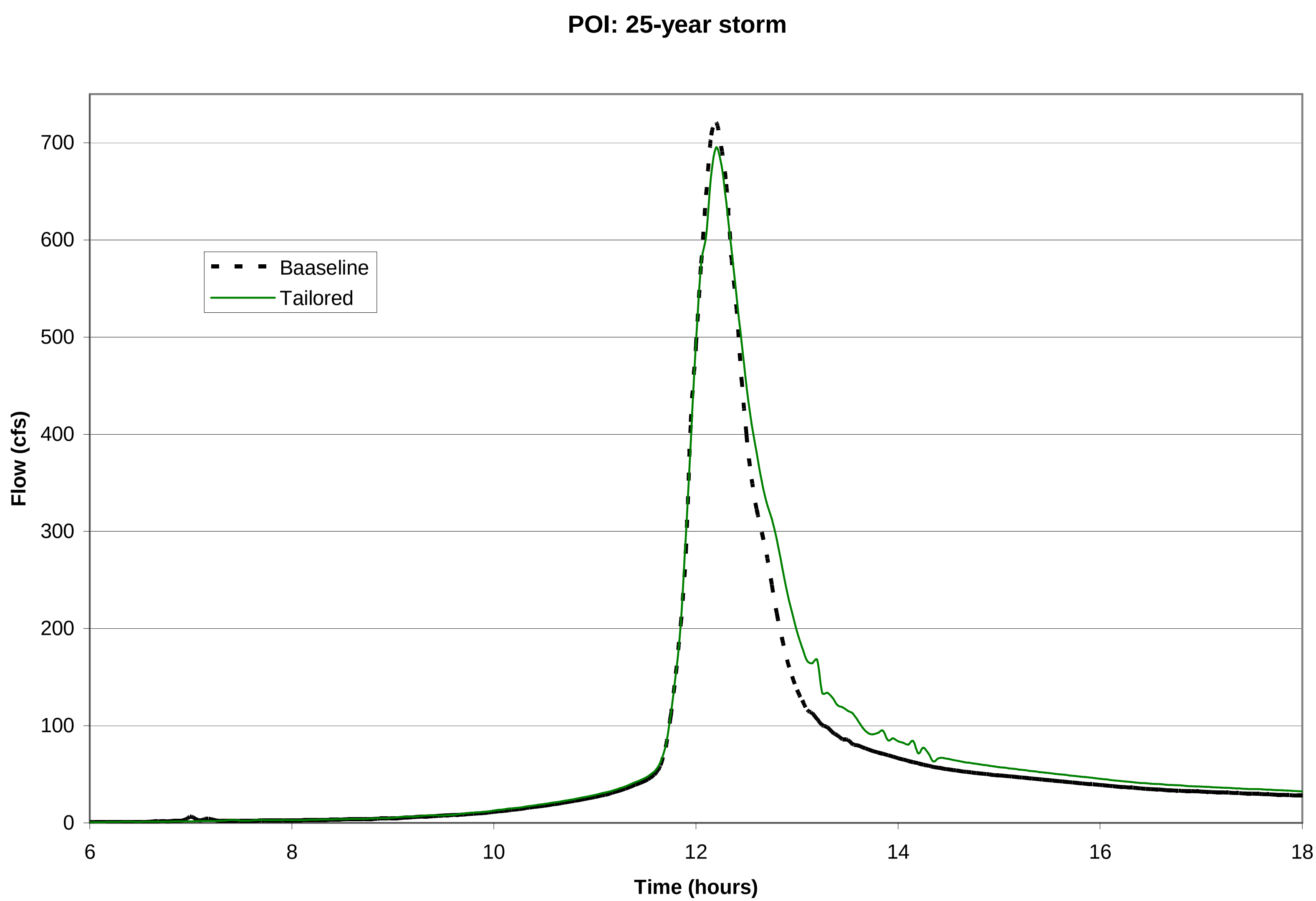
S-1. Comparison of runoff hydrographs for the overall watershed resulting from implementation of the Tailored strategy for the 1-year design storm, compared to the watershed baseline hydrographs.



S-2. Comparison of runoff hydrographs for the overall watershed resulting from implementation of the Tailored strategy for the 2-year design storm, compared to the watershed baseline hydrographs.



S-3. Comparison of runoff hydrographs for the overall watershed resulting from implementation of the Tailored strategy for the 10-year design storm, compared to the watershed baseline hydrographs.



S-4. Comparison of runoff hydrographs for the overall watershed resulting from implementation of the Tailored strategy for the 25-year design storm, compared to the watershed baseline hydrographs.